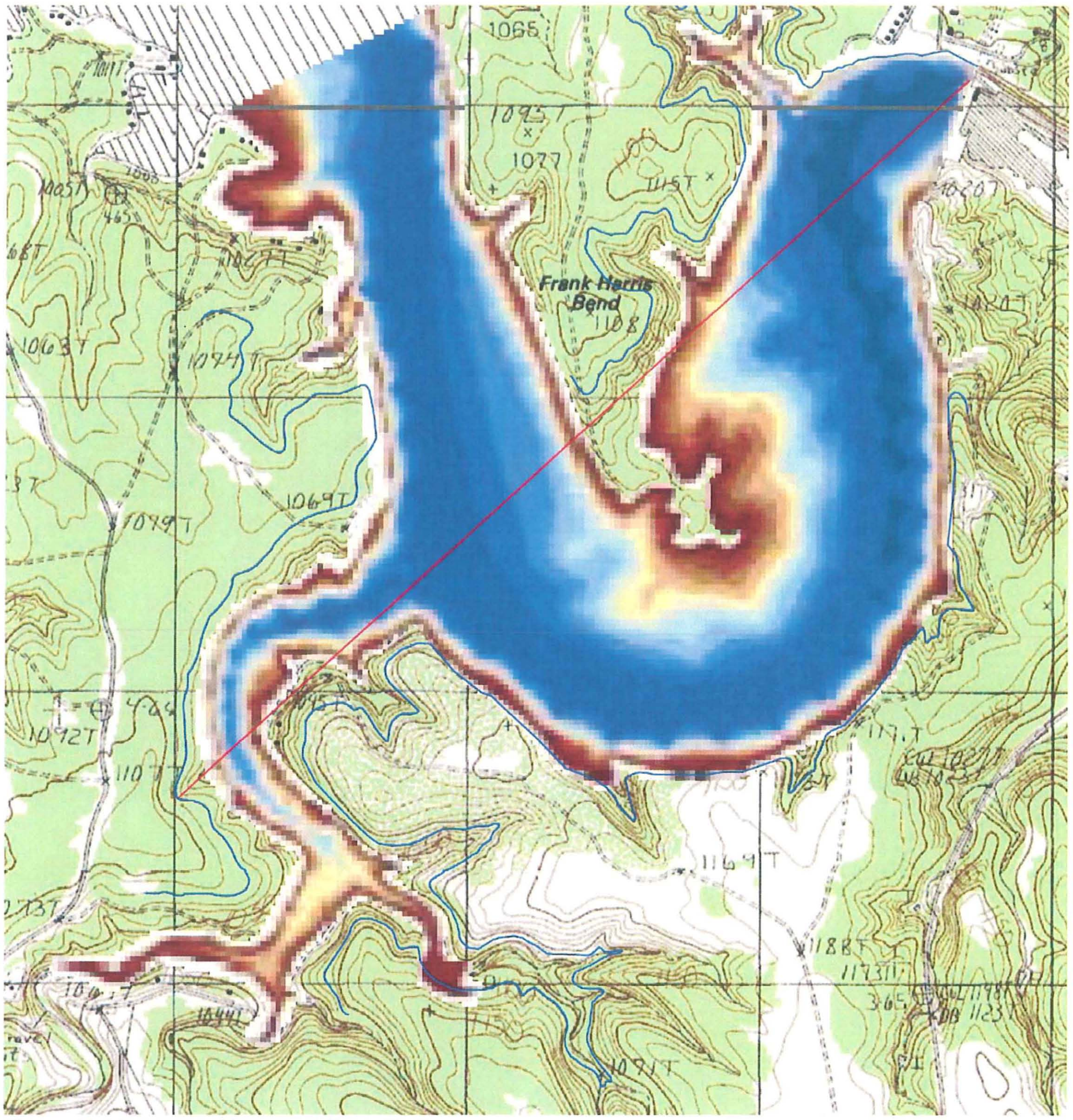


Figure 7-40. Morris Sheppard Dam Bathymetry

The data for the distance and elevations are tabulated in Table 7-15. An average depth along the fetch distance is determined using the data in Table 7-15 and the following formula for hydraulic depth:

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * (X_2 - X_1) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * (X_n - X_{n-1})}{X_n - X_1}$$

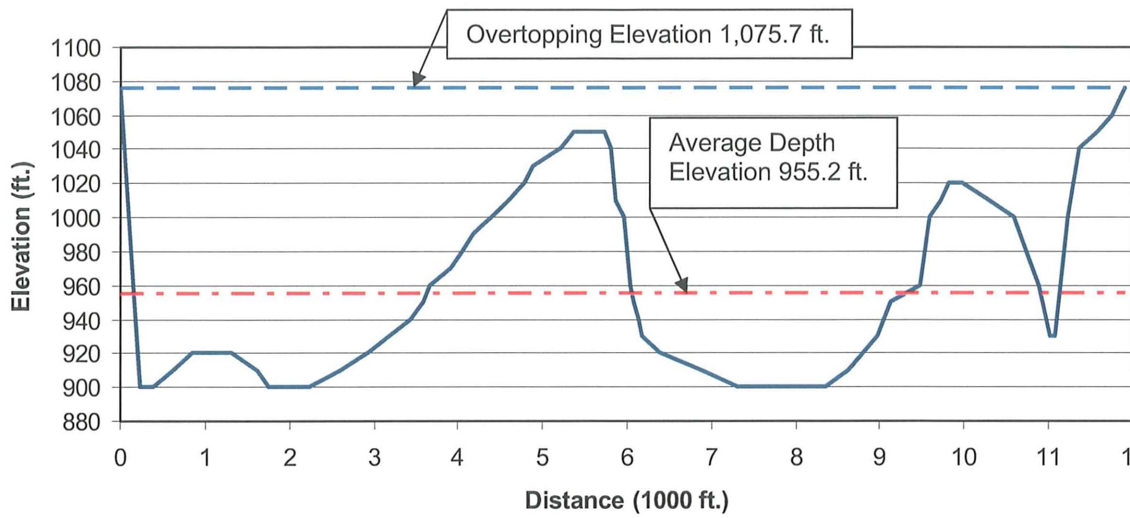


Figure 7-41. Morris Sheppard Dam Bottom Surface Profile

Table 7-15. Morris Sheppard Dam Bottom Surface Profile Section Coordinates

Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)
0	1,075.7	4,395	1,000	8,618	910
227	900	4,601	1,010	8,973	930
390	900	4,781	1,020	9,116	950
627	910	4,886	1,030	9,466	960
847	920	5,209	1,040	9,591	1,000
1,318	920	5,355	1,050	9,724	1,010
1,612	910	5,734	1,050	9,816	1,020
1,746	900	5,799	1,040	9,982	1,020
2,232	900	5,872	1,010	10,596	1,000
2,598	910	5,968	1,000	10,872	960
2,941	920	6,036	960	11,015	930
3,178	930	6,069	950	11,076	930
3,443	940	6,129	940	11,234	1,000
3,593	950	6,177	930	11,371	1,040
3,668	960	6,377	920	11,581	1,050
3,913	970	6,880	910	11,740	1,060
4,053	980	7,320	900	11,844	1,070
4,194	990	8,354	900	11,911	1,075.7

Note: Distance 0 ft. is at the dam.

The average depth bottom surface elevation is calculated to be 955.2 ft. The overtopping water surface elevation is 1,075.7 ft. Therefore, the average depth along the fetch distance is calculated to be 1,075.7 ft. – 955.2 ft. = 120.5 ft. From above, the wind speed is 60 mph and the fetch distance is 2.3 mi. Wind setup is calculated as follows:

$S = (60\ \text{mph})^2 * (2.3\ \text{mi.}) / (1{,}400 * 120.5\ \text{ft.}) = 0.05\ \text{ft.}$

Setup is conservatively rounded up to 0.1 ft. For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. For the 5,420,000 cfs overtopping scenario, the PMF headwater elevation at Morris Sheppard Dam including wind setup is 1,075.7 ft. + 0.1 ft. = 1,075.8 ft. For the 5,120,000 cfs overtopping scenario, the PMF headwater elevation at Morris Sheppard Dam including wind setup is 1,073.3 ft. + 0.1 ft. = 1,073.4 ft.

Dam Failure Morris Sheppard Dam

Four failure scenarios are postulated as described below.

1. Overtopping failure of the spillway section
2. Overtopping failure of the embankment section
3. Overtopping failure of the buttress section at the left abutment
4. Overtopping failure of the buttress section between the spillway and embankment sections

The overtopping failures of the buttress sections are eliminated without calculation. In reference to Figure 7-25, the left abutment buttress section has a shorter crest length than the spillway section. Therefore, failure of the spillway section would result in a greater breach flow. The buttress section between the spillway and embankment sections is approximately the same length as the spillway, but the section depth is about half that of the spillway section. Therefore, failure of the spillway section would result in a greater breach flow.

Figures 7-42, 7-43, and 7-44, show the maximum depth of the embankment section as approximately at elevation 990.0 ft. According to the geotechnical report (Reference 11), the concrete core wall noted in the figures is 2 ft. wide and extends into the foundation. The core wall is assumed to fail with the embankment section due to overtopping. The LSt and SaSt designations in Figure 7-44 correspond to limestone and sandstone respectively.

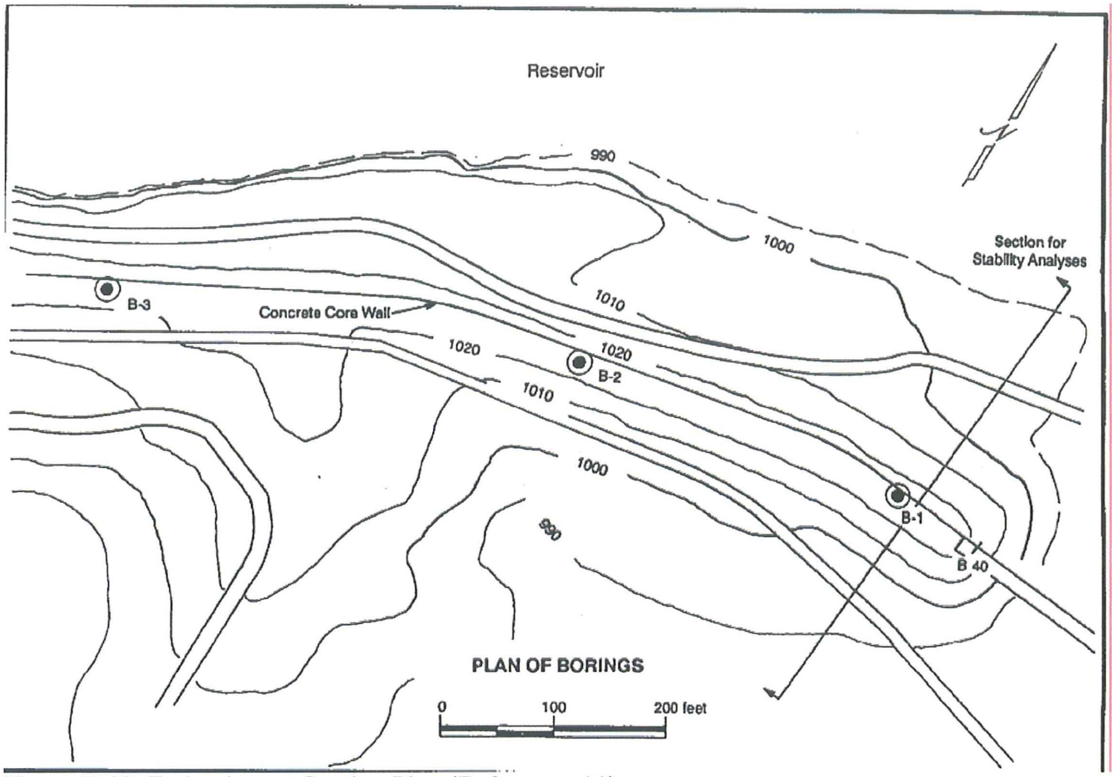


Figure 7-42. Embankment Section Plan (Reference 11)

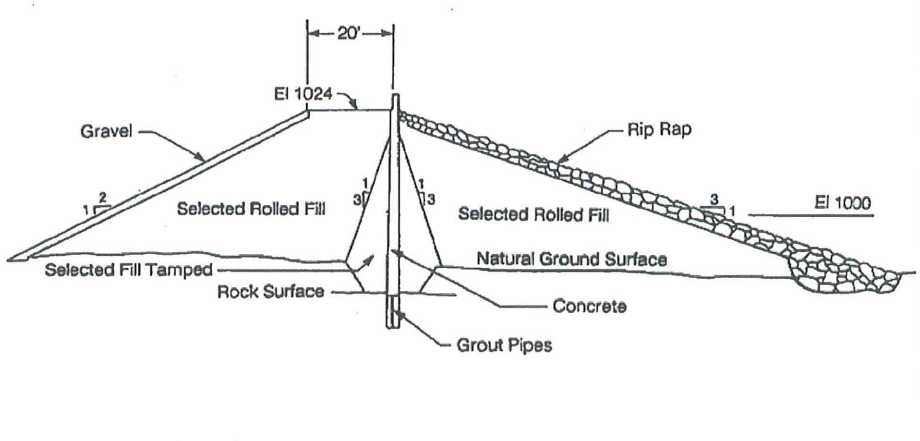
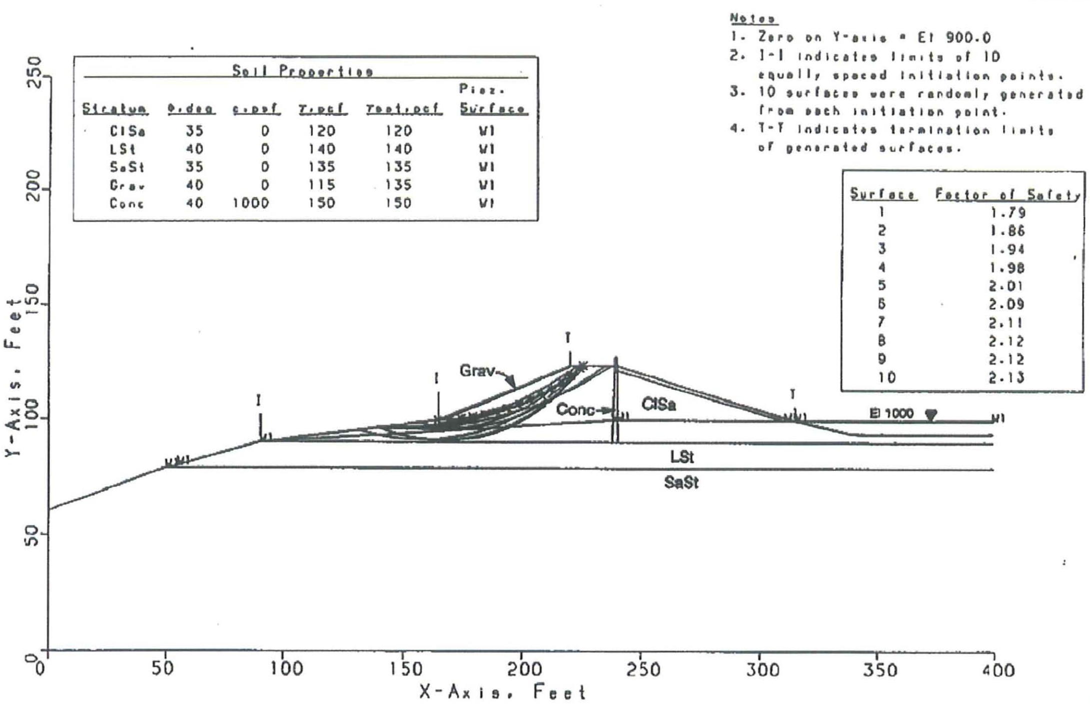


Figure 7-43. Embankment Section Typical Cross Section (Reference 11)



SLOPE STABILITY RESULTS
South Embankment
Normal Pool-El 1000

Figure 7-44. Embankment Section Slope Stability Cross Section (Reference 11)

As previously discussed, dam failure is evaluated based on two methods. As identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2) the breach wave height is computed as h = 4 * (headwater – tailwater) / 9 and transposed downstream without attenuation. Alternatively, dam failure flow is calculated using a USACE dam breach equation (Reference 24) and USACE RD-13 breach parameters (Reference 23).

As identified above, the dam crest elevation is 1,024.0 ft. and the concrete core wall top elevation is 1,028.0 ft. For the 5,420,000 cfs overtopping scenario, the headwater is determined to be 1,075.8 ft. and the tailwater is determined to be 973.0 ft. Because the tailwater is below the toe of the embankment section, the breach wave height is calculated separately for the spillway section. The breach wave height for the spillway section is calculated as follows:

h = 4 * (1,075.8 ft. – 973.0 ft.) / 9 = 45.69 ft., rounded up to 45.7 ft.

As identified above the bottom of the embankment section is at elevation 990.0 ft. The breach wave height for the embankment section is calculated as follows:

h = 4 * (1,075.8 ft. – 990.0 ft.) / 9 = 38.13 ft., rounded up to 38.2 ft.

For the 5,120,000 cfs overtopping scenario, the headwater is determined to be 1,073.4 ft. and the tailwater is determined to be 970.3 ft. Because the tailwater is below the toe of the embankment section, the breach wave height is calculated separately. The breach wave height for the spillway section is calculated as follows:

h = 4 * (1,073.4 ft. – 970.3 ft.) / 9 = 45.82 ft., rounded to 45.9 ft.

As identified above the bottom of the embankment section is at elevation 990.0 ft. The breach wave height for the embankment section is calculated as follows:

h = 4 * (1,073.4 ft. – 990.0 ft.) / 9 = 37.07 ft., rounded to 37.1 ft.

Breach parameters for the embankment section are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. The breach width, W_b, is three times the dam height, and the side slopes of the breach are 1:1 (horizontal:vertical). From above, the embankment section dam height is 1,024.0 ft. – 990.0 ft. = 34.0 ft.

Therefore, W_b = 3 * height of dam = 3 * 34.0 ft. = 102.0 ft.

HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used for the dam break equation including side slopes.

$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. As previously discussed, the tailwater is below the toe of the embankment dam. Therefore the breach bottom is used to determine the water height. For the 5,420,000 cfs overtopping scenario, the water height is 1,075.8 ft. – 990 ft. = 85.8 ft. Therefore the breach flow is calculated as follows:

$Q = 1.7 * 102.0\text{ ft.} * (85.8\text{ ft.})^{1.5} + 1.35 * 1 * (85.8\text{ ft.})^{2.5}$ = 229,866 cfs, rounded up to 230,000 cfs.

The total flow is assumed to be the sum of the overtopping flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 230,000 cfs + 5,420,000 cfs = 5,650,000 cfs.

For the 5,120,000 cfs overtopping scenario, the water height is 1,073.4 ft. – 990 ft. = 83.4 ft. Therefore the breach flow is calculated as follows:

$Q = 1.7 * 102.0\text{ ft.} * (83.4\text{ ft.})^{1.5} + 1.35 * 1 * (83.4\text{ ft.})^{2.5}$ = 217,821 cfs, rounded up to 220,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 220,000 cfs + 5,120,000 cfs = 5,340,000 cfs.

By comparison, the spillway section breach flow is determined by the USACE EM-1110-2-1420 (Reference 22, Page 16-3, equation 16-1) dam break equation.

$Q = (8 / 27) * W_b * g^{0.5} * h^{1.5}$

Breach parameters for the spillway section are determined by USACE RD-13 (Reference 23, Table 1, Page 17). For concrete gravity dams, the breach width is a multiple of the monolith widths. In this case the entire spillway section is assumed to fail. From above, the spillway section is 707 ft. in length. Therefore, the breach width is 707 ft. The side slopes are 0:1 (horizontal:vertical).

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. The spillway section is in the stream bed, which is below the tailwater. Therefore, the tailwater is used to determine the water height. For the 5,420,000 cfs overtopping scenario, the water height is 1,075.8 ft. – 973.0 ft. = 102.8 ft. Therefore the breach flow is calculated as follows:

$Q = (8 / 27) * 707 \text{ ft.} * (32.2 \text{ ft/sec}^2)^{0.5} * (102.8 \text{ ft.})^{1.5}$ = 1,238,977 cfs, rounded up to 1,240,000 cfs.

The total flow is assumed to be the sum of the overtopping flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 1,240,000 cfs + 5,420,000 cfs = 6,660,000 cfs.

For the 5,120,000 cfs overtopping scenario, the water height is 1,073.4 ft. – 970.3 ft. = 103.1 ft. Therefore the breach flow is calculated as follows:

$Q = (8 / 27) * 707 \text{ ft.} * (32.2 \text{ ft/sec}^2)^{0.5} * (103.1 \text{ ft.})^{1.5}$ = 1,244,404 cfs, rounded up to 1,250,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 1,250,000 cfs + 5,120,000 cfs = 6,370,000 cfs.

In summary, the critical potential scenarios for the Morris Sheppard Dam failure effects, including the domino-type and simultaneous failures from upstream dams, transposed downstream without attenuation are determined to be a spillway section breach wave height of 45.9 ft., or a spillway section total breach flow of 6,660,000 cfs. The breach wave height and breach flow are transposed downstream to the De Cordova Bend Dam without any attenuation.

<u>Dam Failure De Cordova Bend Dam</u>

The PMF for the Brazos River was previously determined based on the drainage area at De Cordova Bend Dam. However, the PMF for the Brazos River was applied to the upstream Morris Sheppard Dam. Therefore, the dam failure effects from Morris Sheppard Dam include the PMF to be applied at De Cordova Bend Dam. The Morris Sheppard Dam failure flow, including the Brazos River PMF, is applied to De Cordova Bend Dam without any attenuation. From above, the total flow to determine the water surface elevation is 6,660,000 cfs. Alternatively, to determine the water surface elevation a breach wave height of 45.9 ft. is applied to the De Cordova Bend Dam.

According to the Texas Water Development Board volumetric survey (Reference 19), De Cordova Bend Dam is a concrete buttress dam with earthfill sections and has a maximum height of 84 ft. The total length of the dam is 2,200 ft. The spillway section is gate controlled with an ogee crest elevation of 658.0 ft. There are 16 tainter gates, each 36 ft. wide and 35 ft. high. Therefore, the top of gates elevation is 658.0 ft. + 35 ft. = 693.0 ft. The dam impounds Lake Granbury at a normal pool elevation of 693.0 ft. Based on an earlier volumetric survey (Reference 18), the top of the dam is elevation 706.5 ft. De Cordova Bend Dam is shown in Figure 7-45.

According to the USGS gauge 08090900 Water-Data Report 2008 (Reference 28), the maximum recorded elevation for the reservoir is 693.60 ft.

According to the NID database (Reference 25), the spillway section is 656 ft. long.

Figure 7-45. De Cordova Bend Dam (Reference 5)

As the PMF including the effects of upsteam dam failures is applied, it is assumed that the spillway gates are closed and that Lake Granbury is at the historical maximum recorded elevation of 693.6 ft. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$$Q = C * L * H^{1.5}$$

As previously discussed in Section 6.0, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

Based on USGS quadrangles (Reference 32) and the Morris Sheppard Dam results from above, it becomes apparent that high overtopping flow would spread out at the abutments of De Cordova Bend Dam. Additionally, there are low areas along the south rim of the reservoir that would be susceptible to discharge at high overtopping flows. The 7.5 minute USGS quadrangles (Reference 32) for Nemo, TX and Acton, TX are inserted into AutoCAD (Reference 2) to determine distances and elevations for the overtopping evaluation. Figure 7-46 identifies the selected sections in relationship to the dam. Figure 7-47 shows the section for the dam. Table 7-16 provides the station and elevation information for the section along the dam. Figure 7-48 shows the section for the south rim. Table 7-17 provides the station and elevation information for the section along the south rim.

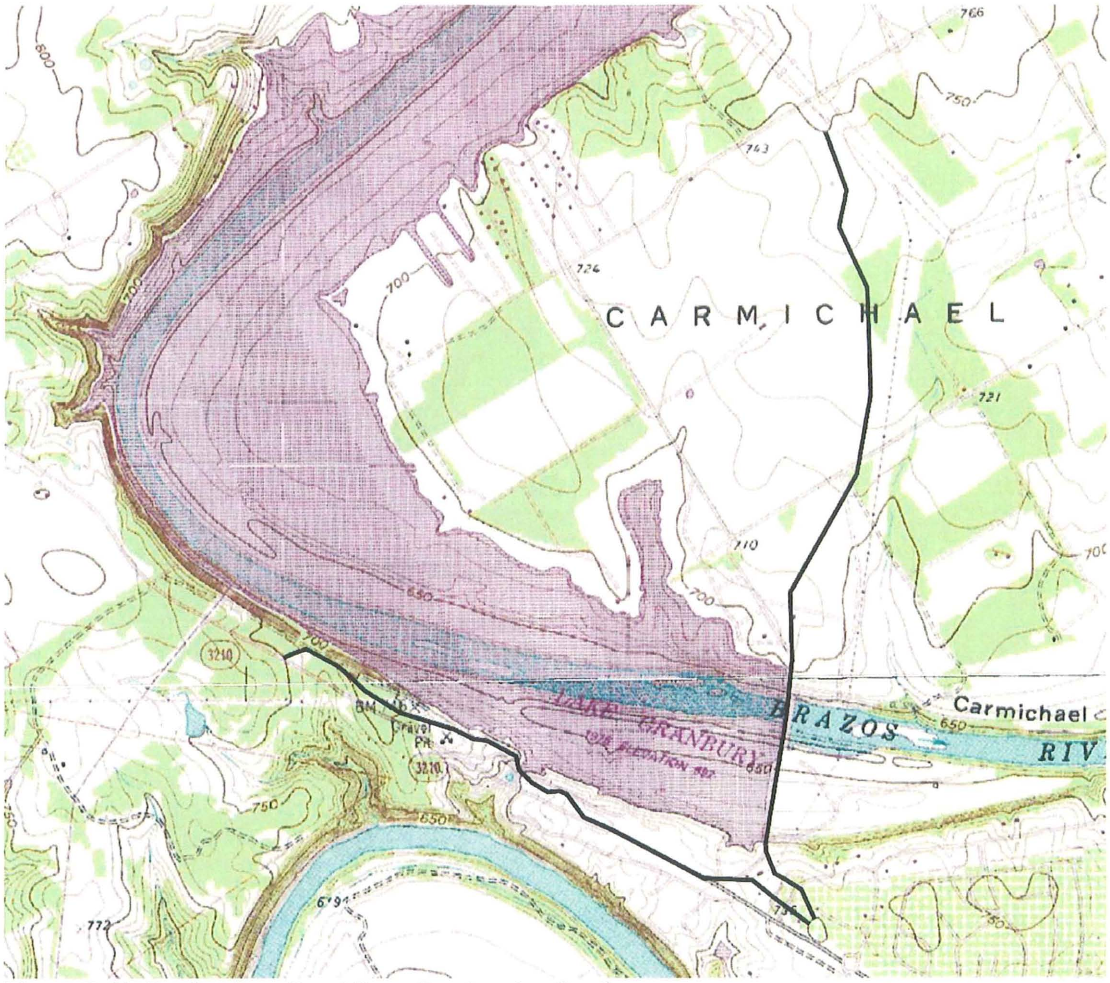


Figure 7-46. De Cordova Bend Dam Overtopping Sections

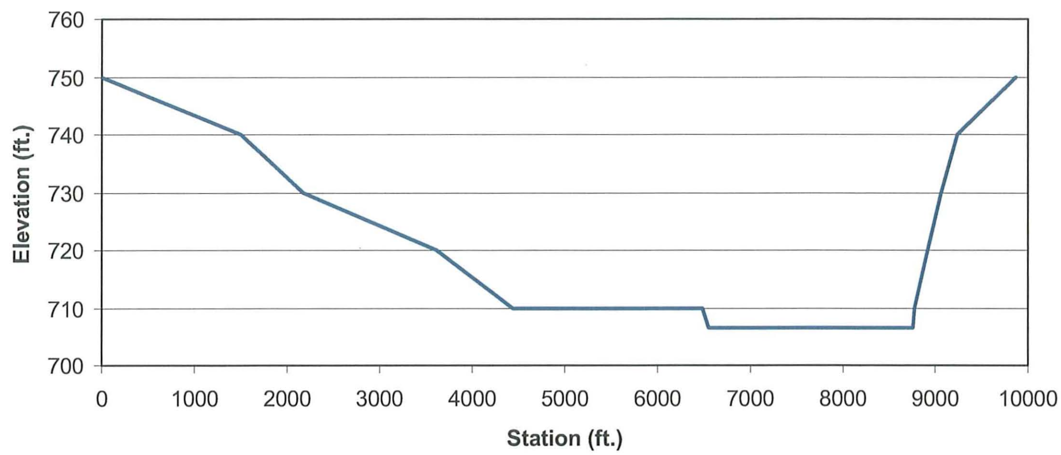


Figure 7-47. De Cordova Bend Dam and Abutments Section

Table 7-16. De Cordova Bend Dam and Abutments Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
0	750	8,754	706.5
1,500	740	8,772	710
2,185	730	8,914	720
3,619	720	9,065	730
4,443	710	9,237	740
6,480	710	9,864	750
6,554	706.5		

Stationing from left to right when looking downstream
The dam is located from station 6,554 ft. to station 8,754 ft.

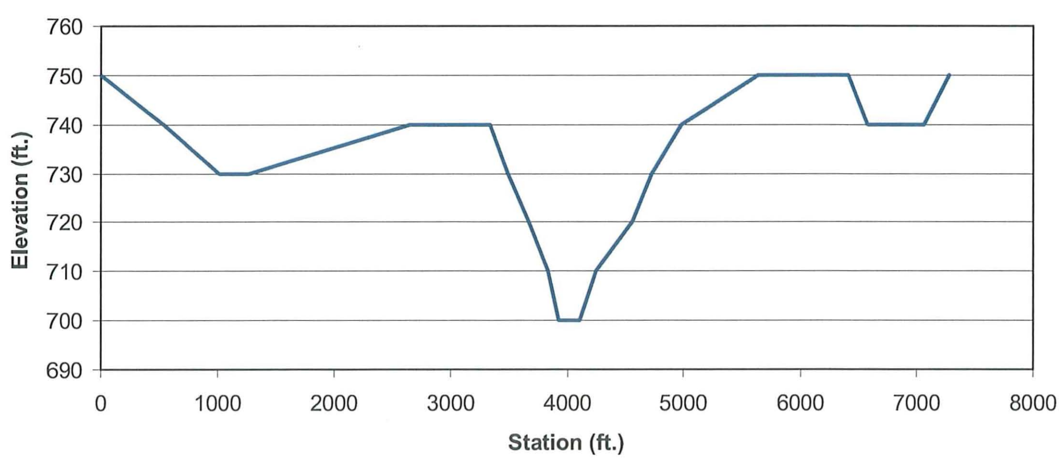


Figure 7-48. De Cordova Bend Dam South Rim Section

Table 7-17. De Cordova Bend Dam South Rim Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
0	750	4,105	700
537	740	4,248	710
1,016	730	4,554	720
1,267	730	4,724	730
2,657	740	4,986	740
3,345	740	5,643	750
3,490	730	6,413	750
3,677	720	6,570	740
3,839	710	7,067	740
3,928	700	7,281	750

Stationing from left to right when looking downstream

For both the dam section and the south rim section, a generalized equation is used to determine the overtopping elevation. The following equation is used for each segment of the sections:

$$Q = C * (L_{x+1} - L_x) * (Z - (E_y + E_{y+1})/2)^{1.5}$$

Where:
Q = flow (cfs)
C = weir flow coefficient
L_x = section station length (ft.)
L_{x+1} = next section station length (ft.)
E_y = elevation at station (ft.)
E_{y+1} = elevation at next station (ft.)
Z = overtopping water surface elevation (ft.)

The dam section and south rim section are simplified as shown below in Figure 7-49 and Figure 7-50, respectively. The resulting water surface elevation shown on the section figures is the determined overtopping elevation as detailed below. The data for the simplified sections are provided in Table 7-18 and Table 7-19. Overtopping is based on the assumption that the reservoir is full up to the crest elevation of 706.5 ft. This assumption exceeds the maximum historical water surface elevation for the reservoir.

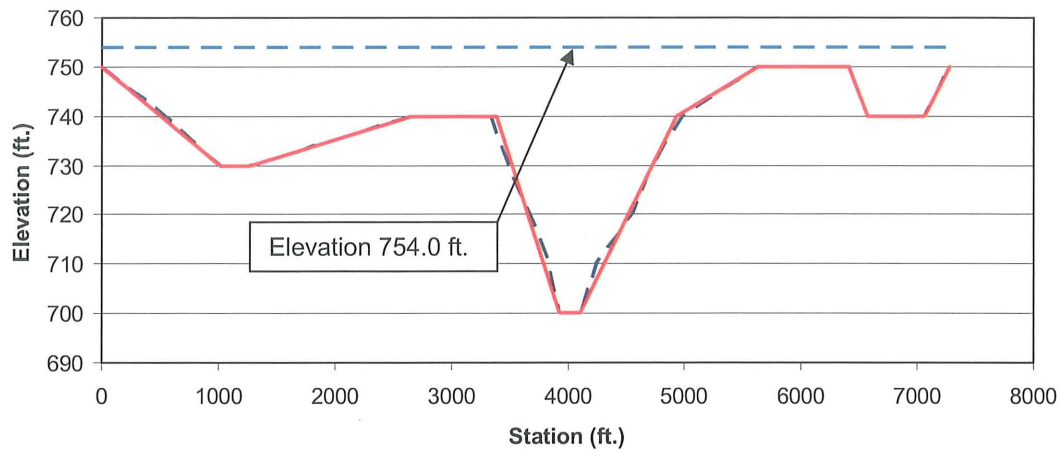


Figure 7-49. De Cordova Bend Dam South Rim Approximated Section 6,660,000 cfs Headwater

Table 7-18. De Cordova Bend Dam South Rim Approximated Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
0	750	8,754	706.5
1,500	740	8,754	710
4,443	710	9,237	740
6,554	710	9,864	750
6,554	706.5	8,754	706.5

Stationing from left to right when looking downstream
The dam is located from station 6,554 ft. to station 8,754 ft.

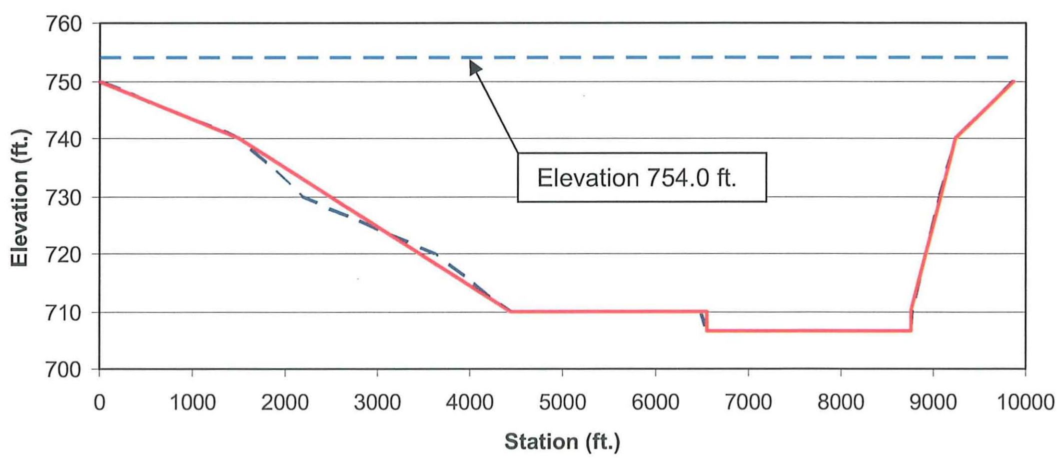


Figure 7-50. De Cordova Bend Dam and Abutments Approximated Section 6,660,000 cfs Headwater

Table 7-19. De Cordova Bend Dam and Abutments Approximated Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
0	750	4,930	740
1,016	730	5,643	750
1,267	730	6,413	750
2,657	740	6,570	740
3,390	740	7,067	740
3,928	700	7,281	750
4,105	700		

Stationing from left to right when looking downstream

The overtopping elevation is determined for the upstream dam failures and combined PMF of 6,660,000 cfs. Using the data above, the combined weir flow equation is:

$$
\begin{aligned}
6{,}660{,}000 \text{ cfs} &= 2.6 * (1{,}500 \text{ ft.} - 0 \text{ ft.}) * (Z - (750 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (4{,}443 \text{ ft.} - 1{,}500 \text{ ft.}) * (Z - (740 \text{ ft.} + 710 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (6{,}554 \text{ ft.} - 4{,}443 \text{ ft.}) * (Z - (710 \text{ ft.} + 710 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (8{,}754 \text{ ft.} - 6{,}554 \text{ ft.}) * (Z - (706.5 \text{ ft.} + 706.5 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (9{,}237 \text{ ft.} - 8{,}754 \text{ ft.}) * (Z - (710 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (9{,}864 \text{ ft.} - 9{,}237 \text{ ft.}) * (Z - (740 \text{ ft.} + 750 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (1{,}016 \text{ ft.} - 0 \text{ ft.}) * (Z - (750 \text{ ft.} + 730 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (1{,}267 \text{ ft.} - 1{,}016 \text{ ft.}) * (Z - (730 \text{ ft.} + 730 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (2{,}657 \text{ ft.} - 1{,}267 \text{ ft.}) * (Z - (730 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (3{,}390 \text{ ft.} - 2{,}657 \text{ ft.}) * (Z - (740 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (3{,}928 \text{ ft.} - 3{,}390 \text{ ft.}) * (Z - (740 \text{ ft.} + 700 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (4{,}105 \text{ ft.} - 3{,}928 \text{ ft.}) * (Z - (700 \text{ ft.} + 700 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (4{,}930 \text{ ft.} - 4{,}105 \text{ ft.}) * (Z - (700 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (5{,}643 \text{ ft.} - 4{,}930 \text{ ft.}) * (Z - (740 \text{ ft.} + 750 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (6{,}413 \text{ ft.} - 5{,}643 \text{ ft.}) * (Z - (750 \text{ ft.} + 750 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (6{,}570 \text{ ft.} - 6{,}413 \text{ ft.}) * (Z - (750 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (7{,}067 \text{ ft.} - 6{,}570 \text{ ft.}) * (Z - (740 \text{ ft.} + 740 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (7{,}281 \text{ ft.} - 7{,}067 \text{ ft.}) * (Z - (740 \text{ ft.} + 750 \text{ ft.})/2)^{1.5}
\end{aligned}
$$

Solving for overtopping elevation, Z = 753.96 ft. = 754.0 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. As shown above the elevation exceeds the horizontal limits of the section. The calculation remains conservative because flow is not permitted to spread out horizontally, resulting in a higher headwater elevation.

Using the overtopping elevation 753.96 ft., the portion of the flow that overtops the dam is determined to be 5,006,475 cfs, rounded down to 5,000,000 cfs. This flow is used to determine the tailwater effects. Therefore, rounding down is conservative because it maximizes the water height component of the dam failure equation and the resulting dam failure flow.

Alternatively, a breach wave height of 45.9 ft. is applied to the De Cordova Bend Dam. The PMF for De Cordova Bend Dam has been incorporated into the Morris Sheppard Dam failure scenario as previously described. A breach wave height of 45.9 ft. results in an overtopping elevation of 693.6 ft.

+ 45.9 ft. = 739.5 ft. The overtopping flow is determined using the dam section and south rim section identified above. However, to determine flow over the spillway of the dam section, the top elevation of the gates, 693 ft., is used for the bottom of the section. This is conservative because it allows more flow to pass through the section than if the crest of the dam is used. Therefore, because there are 16 gates, 36 ft. wide, the length used is 16 * 36 ft. = 576.0 ft.

The dam section and south rim section are simplified as shown below in Figure 7-51 and Figure 7-52, respectively. The data for the simplified sections are provided in Table 7-20 and Table 7-21. Because the overtopping elevation does not exceed the entire south rim section, there are only two areas where overtopping flow occurs, as shown.

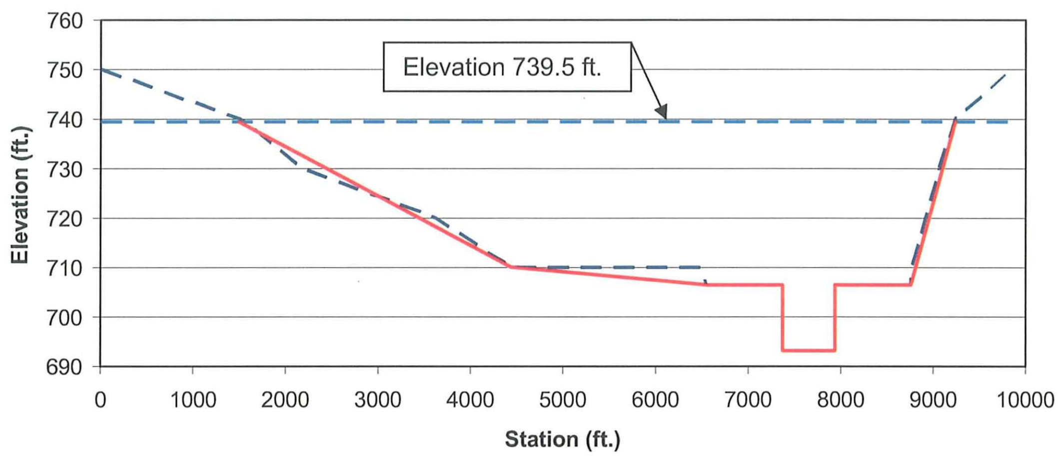


Figure 7-51. De Cordova Bend Dam and Abutments Approximated Section Elevation 739.5 ft. Headwater

Table 7-20. De Cordova Bend Dam and Abutments Approximated Section Elevation 739.5 ft. Headwater Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
1,500	739.5	7,942	693
4,443	710	7,942	706.5
6,554	706.5	8,754	706.5
7,366	706.5	9,237	739.5
7,366	693		

Stationing from left to right when looking downstream
The dam is located from station 6,554 ft. to station 8,754 ft.
The top of gate is represented by station 7,366 ft. to station 7,942 ft.

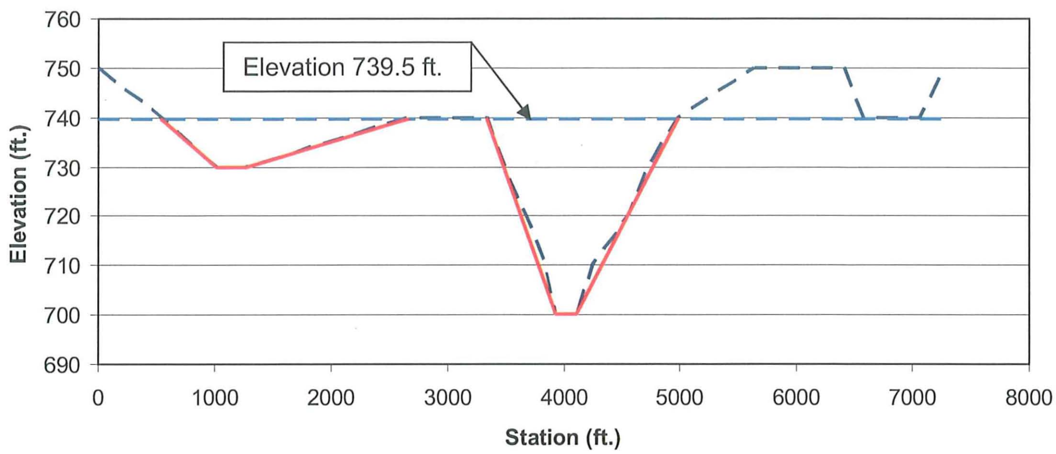


Figure 7-52. De Cordova Bend Dam South Rim Approximated Section Elevation 739.5 ft. Headwater Coordinates

Table 7-21. De Cordova Bend Dam South Rim Approximated Section Elevation 739.5 ft. Headwater Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
537	739.5	3,345	739.5
1,016	730	3,928	700
1,267	730	4,105	700
2,657	739.5	4,986	739.5

Stationing from left to right when looking downstream

The overtopping flow is determined for the breach wave height representing the effects of the upstream dam failures. Using the data above, the combined weir flow equation is:

$$
\begin{aligned}
Q = &\ 2.6 * (4{,}443 \text{ ft.} - 1{,}500 \text{ ft.}) * (Z - (739.5 \text{ ft.} + 710 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (6{,}554 \text{ ft.} - 4{,}443 \text{ ft.}) * (Z - (710 \text{ ft.} + 706.5 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (7{,}366 \text{ ft.} - 6{,}554 \text{ ft.}) * (Z - (706.5 \text{ ft.} + 706.5 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (7{,}942 \text{ ft.} - 7{,}366 \text{ ft.}) * (Z - (693 \text{ ft.} + 693 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (8{,}754 \text{ ft.} - 7{,}942 \text{ ft.}) * (Z - (693 \text{ ft.} + 706.5 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (9{,}237 \text{ ft.} - 8{,}754 \text{ ft.}) * (Z - (706.5 \text{ ft.} + 739.5 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (1{,}016 \text{ ft.} - 537 \text{ ft.}) * (Z - (739.5 \text{ ft.} + 730 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (1{,}267 \text{ ft.} - 1{,}016 \text{ ft.}) * (Z - (730 \text{ ft.} + 730 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (2{,}657 \text{ ft.} - 1{,}267 \text{ ft.}) * (Z - (730 \text{ ft.} + 739.5 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (3{,}928 \text{ ft.} - 3{,}345 \text{ ft.}) * (Z - (739.5 \text{ ft.} + 700 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (4{,}105 \text{ ft.} - 3{,}928 \text{ ft.}) * (Z - (700 \text{ ft.} + 700 \text{ ft.})/2)^{1.5} \\
&+ 2.6 * (4{,}986 \text{ ft.} - 4{,}105 \text{ ft.}) * (Z - (700 \text{ ft.} + 739.5 \text{ ft.})/2)^{1.5}
\end{aligned}
$$

Q = 3,269,515 cfs rounded up to 3,270,000 cfs.

The portion of the flow that overtops the dam is determined to be 2,751,763 cfs, rounded down to 2,750,000 cfs. This flow is used to determine the tailwater effects. Therefore, rounding down is conservative because it maximizes the water height component of the dam failure equation and the resulting dam failure flow.

Tailwater is determined for both the transposed breach tailwater flow of 5,000,000 cfs and the transposed breach wave height corresponding to a tailwater flow of 2,750,000 cfs. The 7.5 minute USGS quadrangles (Reference 32) for Nemo, TX and Acton, TX are inserted into AutoCAD (Reference 2) to determine channel distances, slope, and cross section elevations. Figure 7-53 identifies the selected cross section in relationship to the dam and the channel distances used to determine the slope and elevations.

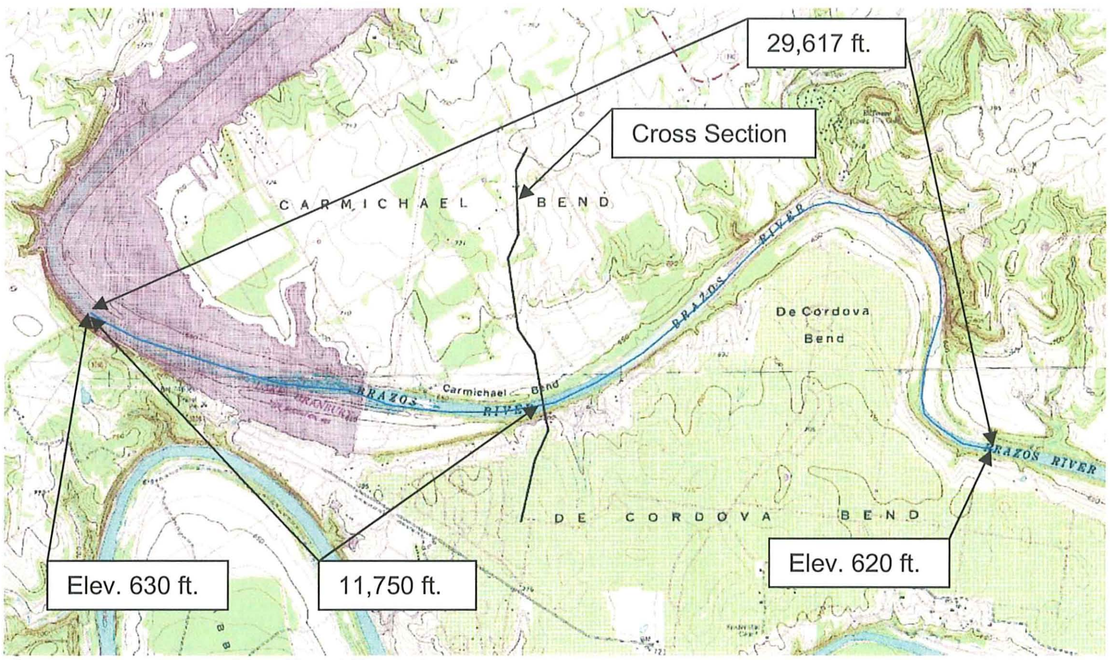


Figure 7-53. De Cordova Bend Dam Downstream

As shown in Figure 7-53, the channel drops 10 ft. over a distance of 29,617 ft. Therefore, the channel slope is 10 ft. / 29,617 ft. = 0.00034 ft./ft., rounded up to 0.0004 ft./ft. The cross section is 11,750 ft. downstream from elevation 630 ft. Therefore, the cross section bottom is 11,750 ft. / 29,617 ft. * 10 ft. = 4 ft. lower than elevation 630 ft. The cross section station and elevations are provided in Table 7-22.

Table 7-22. De Cordova Bend Dam Tailwater Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
-6,674	760	-187	640
-6,481	750	-150	630
-4,425	740	-113	626
-3,528	730	140	626
-2,942	720	167	630
-2,623	710	243	650
-2,388	700	341	700
-1,777	690	543	710
-1,310	680	776	720
-767	670	1,914	730
-584	660	3,047	740
-466	650		

Stationing from left to right when looking downstream

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula. From above, the two flows of 5,000,000 cfs and 2,750,000 cfs were examined with a slope of 0.0004 ft./ft. As previously discussed in Section 6.0, the Manning coefficient of 0.025 is applied to the channel and overbank areas.

The 5,000,000 cfs flow depth for the cross section is determined to be 126.79 ft., rounded down to 126.7 ft. The 2,750,000 cfs flow depth for the cross section is determined to be 108.29 ft. and is rounded down to 108.2 ft. Rounding down is conservative because it results in a lower tailwater elevation as discussed above. The FlowMaster results are provided in Appendix G. Therefore, the tailwater elevation at the downstream cross section is 626 ft. + 126.7 ft. = 752.7 ft. for a flow of 5,000,000 cfs and 626 ft. + 108.2 ft. = 734.2 ft. for a flow of 2,750,000 cfs. Level pool from the cross section upstream to the dam is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. The tailwater elevation in both cases exceeds the dam crest elevation of 706.5 ft. Therefore, the overtopping discharge may be affected by the tailwater. The cross section and tailwater elevations are shown on Figure 7-54.

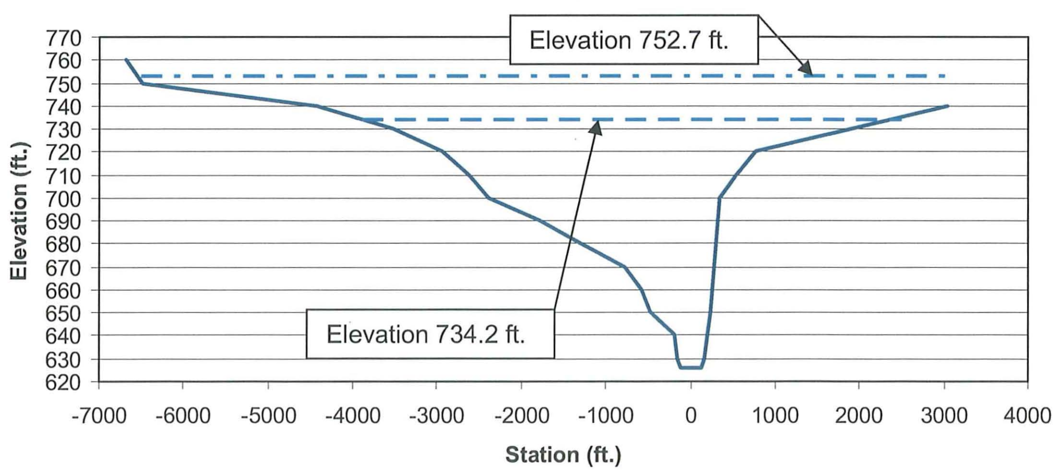


Figure 7-54. De Cordova Bend Dam Tailwater 1st Iteration

The tailwater effects on the headwater elevation are determined using the Federal Highway Administration guidance for roadway overtopping contained in Hydraulic Design Series Number 5 (Reference 14). The weir flow coefficient used to determine the overtopping elevation and flow is modified as necessary using the charts shown on Figure 7-55.

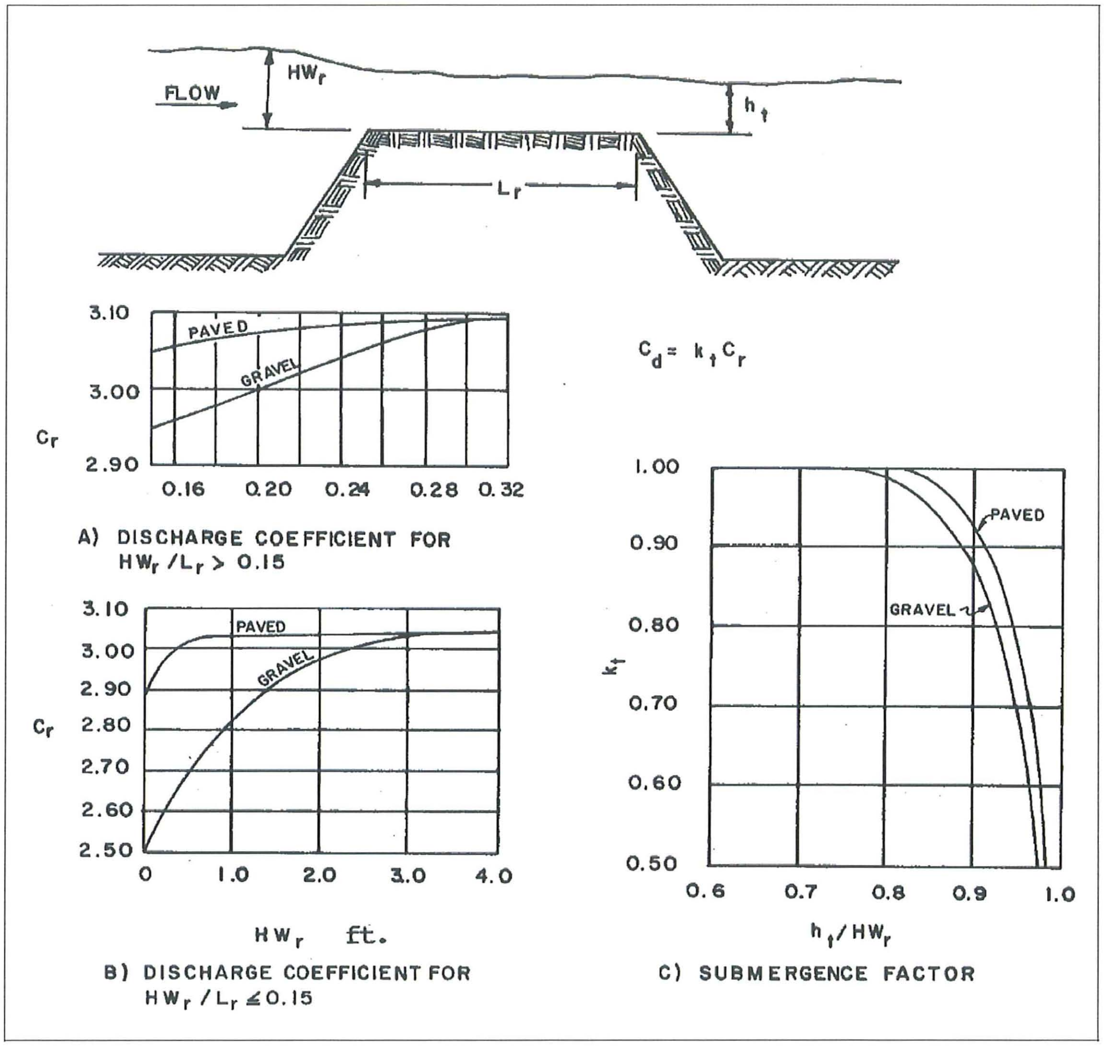


Figure III-11--English Discharge Coefficients for Roadway Overtopping

Figure 7-55. Overtopping Weir Flow Coefficient (Reference 14)

For the case of an overtopping flow of 6,660,000 cfs, the headwater elevation is determined to be 754.0 ft. and the tailwater elevation is determined to be 752.7 ft. The crest elevation is 706.5 ft. According to the Texas Water Development Board volumetric survey (Reference 19), the crest of De Cordova Bend Dam is 17 ft. wide. Therefore, the headwater (754.0 ft. – 706.5 ft. = 47.5 ft.) is much greater than the width of the dam crest. The tailwater is 752.7 ft. – 706. 5 ft. = 46.2 ft. The ratio of tailwater / headwater is 46.2 ft. / 47.5 ft. = 0.97. From Figure 7-55 (A), Cr = 3.09. From Figure 7-55 (C), kt = 0.50. Therefore, the reduced weir flow coefficient C = 3.09 * 0.50 = 1.545, rounded down to 1.54. For the purpose of dam failure evaluation, it is more conservative to use a lower value because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

Using the cross section and data above with the revised weir flow coefficient, the combined weir flow equation is:

6,660,000 cfs = 1.54 * (1,500 ft. – 0 ft.) * (Z – (750 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (4,443 ft. – 1,500 ft.) * (Z – (740 ft. + 710 ft.)/2)$^{1.5}$
+ 1.54 * (6,554 ft. – 4,443 ft.) * (Z – (710 ft. + 710 ft.)/2)$^{1.5}$
+ 1.54 * (8,754 ft. – 6,554 ft.) * (Z – (706.5 ft. + 706.5 ft.)/2)$^{1.5}$
+ 1.54 * (9,237 ft. – 8,754 ft.) * (Z – (710 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (9,864 ft. – 9,237 ft.) * (Z – (740 ft. + 750 ft.)/2)$^{1.5}$
+ 1.54 * (1,016 ft. – 0 ft.) * (Z – (750 ft. + 730 ft.)/2)$^{1.5}$
+ 1.54 * (1,267 ft. – 1,016 ft.) * (Z – (730 ft. + 730 ft.)/2)$^{1.5}$
+ 1.54 * (2,657 ft. – 1,267 ft.) * (Z – (730 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (3,390 ft. – 2,657 ft.) * (Z – (740 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (3,928 ft. – 3,390 ft.) * (Z – (740 ft. + 700 ft.)/2)$^{1.5}$
+ 1.54 * (4,105 ft. – 3,928 ft.) * (Z – (700 ft. + 700 ft.)/2)$^{1.5}$
+ 1.54 * (4,930 ft. – 4,105 ft.) * (Z – (700 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (5,643 ft. – 4,930 ft.) * (Z – (740 ft. + 750 ft.)/2)$^{1.5}$
+ 1.54 * (6,413 ft. – 5,643 ft.) * (Z – (750 ft. + 750 ft.)/2)$^{1.5}$
+ 1.54 * (6,570 ft. – 6,413 ft.) * (Z – (750 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (7,067 ft. – 6,570 ft.) * (Z – (740 ft. + 740 ft.)/2)$^{1.5}$
+ 1.54 * (7,281 ft. – 7,067 ft.) * (Z – (740 ft. + 750 ft.)/2)$^{1.5}$

Solving for overtopping elevation, Z = 766.39 ft. = 766.4 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. The dam section and south rim section are shown with the revised headwater elevation in Figure 7-56 and Figure 7-57, respectively. As shown, the elevation exceeds the horizontal limits of the section. The calculation remains conservative because flow is not permitted to spread out horizontally, resulting in a higher headwater elevation.

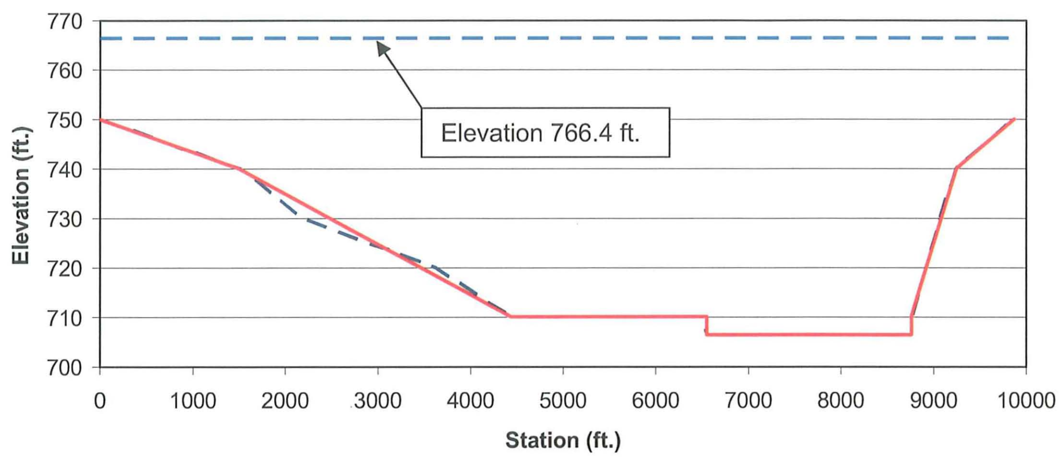


Figure 7-56. De Cordova Bend Dam and Abutments Section 6,660,000 cfs Revised Headwater

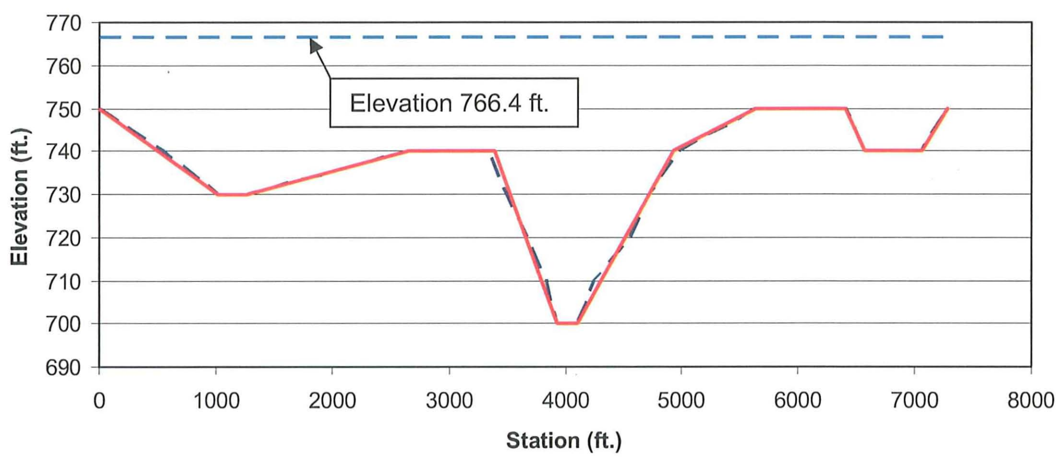


Figure 7-57. De Cordova Bend Dam South Rim Section 6,660,000 cfs Revised Headwater

Using the overtopping elevation 766.39 ft., the portion of the flow that overtops the dam is determined to be 4,675,849 cfs, rounded down to 4,670,000 cfs. This flow is used to determine the revised tailwater effects.

For the case of an overtopping flow of 3,270,000 cfs, the headwater elevation is determined to be 739.5 ft. and the tailwater elevation is determined to be 734.2 ft. From above, the crest elevation is 706.5 ft. and the crest width is 17 ft. Therefore, the headwater (739.5 ft. – 706.5 ft. = 33.0 ft.) is much greater than the width of the dam crest. The tailwater is 734.2 ft. – 706. 5 ft. = 27.7 ft. The ratio of tailwater / headwater is 27.7 ft. / 33.0 ft. = 0.84. From Figure 7-55 (A), Cr = 3.09. From Figure 7-55 (C), kt = 0.95, using the more conservative gravel condition. Therefore, the reduced weir flow coefficient C = 3.09 * 0.95 = 2.9355. This result exceeds the previously used weir flow coefficient of 2.6. Therefore, the previous results are more conservative and remain applicable.

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula for the revised flow of 4,670,000 cfs. As above, the slope is 0.0004 ft./ft. and a Manning coefficient of 0.025 was applied to the channel and overbank areas.

The 4,670,000 cfs flow depth for the cross section is determined to be 125.19 ft. and is rounded down to 125.1 ft. Rounding down is conservative because it results in a lower tailwater elevation as discussed above. The FlowMaster results are provided in Appendix H. Therefore, the tailwater elevation at the downstream cross section is 626 ft. + 125.1 ft. = 751.1 ft. The cross section and revised tailwater elevations are shown on Figure 7-58.

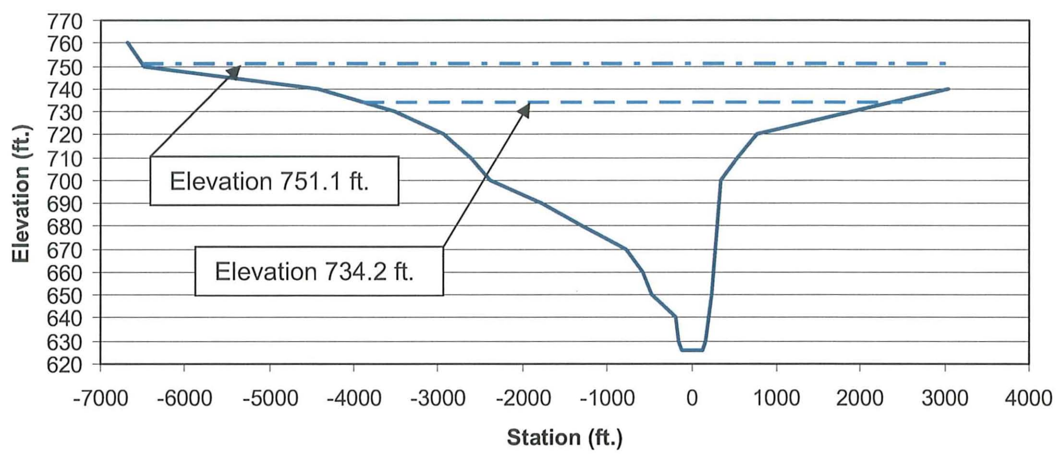


Figure 7-58. De Cordova Bend Dam Tailwater 2nd Iteration

For the case of a total overtopping flow of 6,660,000 cfs, the revised tailwater elevation remains above the dam crest elevation of 706.5 ft. Tailwater effects are reassessed for the revised tailwater elevation. The revised headwater elevation is determined to be 766.4 ft. and the revised tailwater elevation is determined to be 751.1 ft. From above, the crest elevation is 706.5 ft. and the crest width is 17 ft. Therefore, the headwater (766.4 ft. – 706.5 ft. = 59.9 ft.) is much greater than the width of the dam crest. The tailwater is 751.1 ft. – 706. 5 ft. = 44.6 ft. The ratio of tailwater / headwater is 44.6 ft. / 59.9 ft. = 0.74. From Figure 7-55 (A), Cr = 3.09. From Figure 7-55 (C), kt = 1.00. Therefore, the revised weir flow coefficient C = 3.09 * 1.00 = 3.09. This result exceeds the previously used weir flow coefficient of 1.54. Therefore, the previous results are more conservative and remain applicable.

Of the two scenarios examined, the breach flow resulting in an total overtopping flow of 6,660,000 cfs creates the higher headwater elevation. Wind setup for both scenarios is based on the higher headwater elevation which will produce the longer fetch distance.

According to USACE EM 1110-2-1420 (Reference 22) wind setup can be reasonably estimated for lakes and reservoirs using the following equation:

$S = U^2 * F / (1,400 * D)$

USACE EM 1110-2-1420 (Reference 22) indicates that the fetch distance is usually satisfactorily assumed to be two times the effective fetch distance. A straight line fetch is used to define the wind setup and is more conservative than an effective fetch.

As previously discussed, ANSI/ANS-2.8-1992 (Reference 1) is used to define the coincident wind speed. From Figure 7-5, the Annual Extreme-Mile, 30 ft. Above Ground, 2-yr. Mean Recurrence Interval is between 50 mph and 60 mph for the Brazos River watershed upstream from Whitney Lake. The more conservative wind speed of 60 mph is used to generate wind setup.

The controlling overtopping elevation at De Cordova Bend Dam is determined to be 766.4 ft. The fetch length is determined from the reservoir surface area at the overtopping elevation. The 7.5 minute USGS quadrangles (Reference 32) for Nemo, TX and Acton, TX are inserted into AutoCAD (Reference 2) and because only contours with 10 ft. intervals are identified on the quadrangles the 770 ft. contour is used to determine the surface area. As shown on Figure 7-59, the longest straight line fetch distance is determined to be 27,939 ft. (rounded to 5.3 mi.).

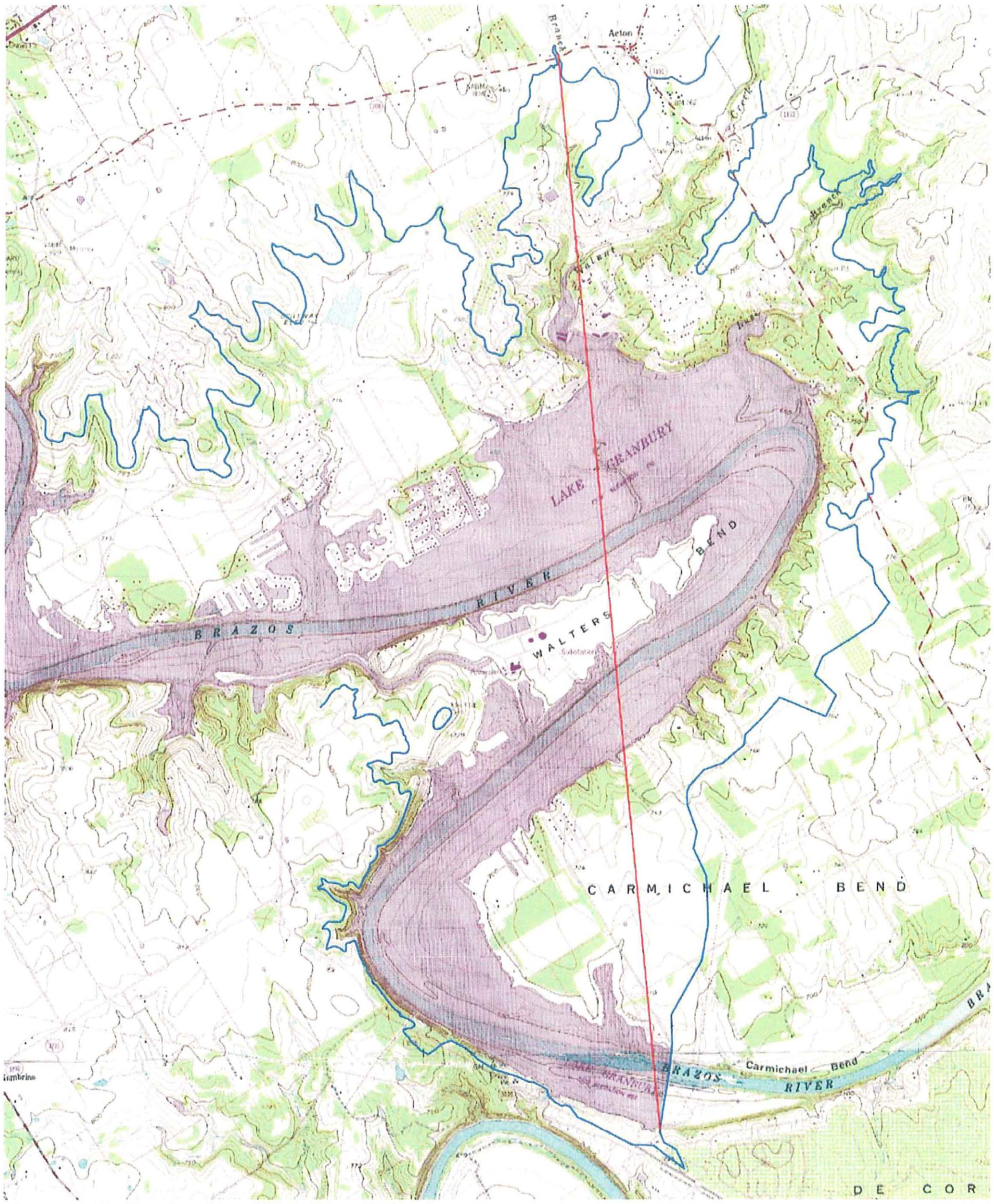


Figure 7-59. De Cordova Bend Dam Fetch Length

A bottom surface profile along the fetch distance is created using the USGS quadrangles (Reference 32) and is provided in Figure 7-60. The data for the distance and elevations are tabulated in Table 7-23. An average depth along the fetch distance is determined using the data in Table 7-23 and the following formula for hydraulic depth:

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * (X_2 - X_1) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * (X_n - X_{n-1})}{X_n - X_1}$$

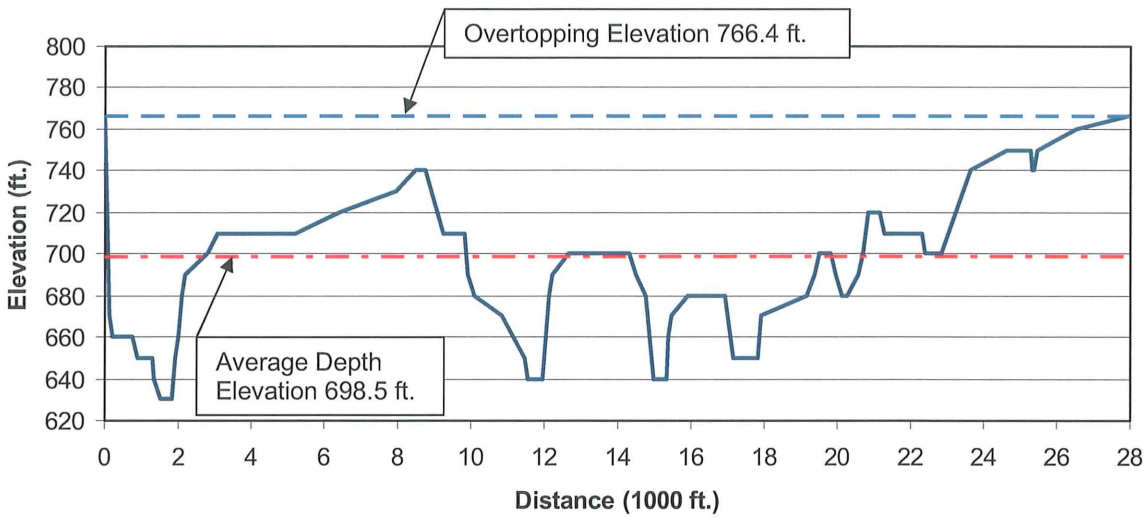


Figure 7-60. De Cordova Bend Dam Bottom Surface Profile

Table 7-23. De Cordova Bend Dam Bottom Surface Profile Section Coordinates

Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)
0	766.4	10,071	680	19,500	700
123	670	10,866	670	19,839	700
233	660	11,485	650	19,977	690
742	660	11,564	640	20,146	680
891	650	11,957	640	20,267	680
1,295	650	12,119	680	20,576	690
1,355	640	12,233	690	20,695	700
1,504	630	12,682	700	20,824	720
1,801	630	14,322	700	21,147	720
1,890	650	14,508	690	21,291	710
2,009	660	14,757	680	22,323	710
2,090	680	14,961	640	22,411	700
2,173	690	15,328	640	22,843	700
2,811	700	15,389	660	23,663	740
3,054	710	15,474	670	24,635	750
5,222	710	15,921	680	25,288	750
6,378	720	16,913	680	25,355	740
7,945	730	17,141	650	25,382	740
8,500	740	17,481	650	25,475	750
8,742	740	17,840	650	26,512	760
9,245	710	17,927	670	27,939	766.4
9,823	710	19,160	680		
9,906	690	19,382	690		

Note: Distance 0 ft. is at the dam.

The average depth bottom surface elevation is calculated to be 698.5 ft. The overtopping water surface elevation is 766.4 ft. Therefore, the average depth along the fetch distance is calculated to be 766.4 ft. – 698.5 ft. = 67.9 ft. From above, the wind speed is 60 mph and the fetch distance is 5.3 mi. Wind setup is calculated as follows:

S = $(60 \text{ mph})^2$ * (5.3 mi.) / (1,400 * 67.9 ft.) = 0.201 ft., rounded up to 0.3 ft.

Setup is conservatively rounded up to 0.3 ft. For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. For the 6,660,000 cfs total overtopping scenario, the headwater elevation at De Cordova Bend Dam including wind setup is 766.4 ft. + 0.3 ft. = 766.7 ft. For the 3,270,000 cfs total overtopping scenario, the headwater elevation at De Cordova Bend Dam including wind setup is 739.5 ft. + 0.3 ft. = 739.8 ft.

There are two postulated failure scenarios, failure of the embankment section, or failure of the spillway section.

As previously discussed, dam failure is evaluated based on two methods. As identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2) the breach wave height is computed as h = 4 * (headwater – tailwater) / 9 and transposed downstream without attenuation. Alternatively, dam failure flow is calculated using a USACE dam breach equation (Reference 24) and USACE RD-13 breach parameters (Reference 23).

As identified above, the dam is 84 ft. tall with a crest elevation at 706.5 ft. For the 6,660,000 cfs total overtopping scenario, the headwater is determined to be 766.7 ft. and the tailwater is determined to be 751.1 ft. The breach wave height is the same for both the embankment and spillway sections and is calculated as follows:

h = 4 * (766.7 ft. – 751.1 ft.) / 9 = 6.93 ft., rounded up to 7.0 ft.

For the 3,270,000 cfs total overtopping scenario, the headwater is determined to be 739.8 ft. and the tailwater is determined to be 734.2 ft. The breach wave height is the same for both the embankment and spillway sections and is calculated as follows:

h = 4 * (739.8 ft. – 734.2 ft.) / 9 = 2.49 ft., rounded up to 2.5 ft.

Breach parameters for the embankment section are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. The breach width, W_b, is three times the dam height, and the side slopes of the breach are 1:1 (horizontal:vertical). From above the maximum dam height is 84 ft.

Therefore, W_b = 3 * height of dam = 3 * 84 ft. = 252 ft.

HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used for the dam break equation including side slopes.

$$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$$

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. For the 6,660,000 cfs total overtopping scenario, the water height is 766.7 ft. – 751.1 ft. = 15.6 ft. Therefore, the breach flow is calculated as follows:

$Q = 1.7 * 252.0 \text{ ft.} * (15.6 \text{ ft.})^{1.5} + 1.35 * 1 * (15.6 \text{ ft.})^{2.5}$ = 27,694 cfs, rounded up to 30,000 cfs.

The total flow is assumed to be the sum of the overtopping flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 30,000 cfs + 6,660,000 cfs = 6,690,000 cfs.

For the 3,270,000 cfs total overtopping scenario, the water height is 739.8 ft. – 734.2 ft. = 5.6 ft. Therefore the breach flow is calculated as follows:

$Q = 1.7 * 252.0 \text{ ft.} * (5.6 \text{ ft.})^{1.5} + 1.35 * 1 * (5.6 \text{ ft.})^{2.5}$ = 5,777 cfs, rounded up to 10,000 cfs.

The total flow is assumed to be the sum of the overtopping flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 10,000 cfs + 3,270,000 cfs = 3,280,000 cfs.

By comparison, the spillway section breach flow is determined using the USACE EM-1110-2-1420 (Reference 22, Page 16-3, equation 16-1) dam break equation.

$Q = (8 / 27) * W_b * g^{0.5} * h^{1.5}$

Breach parameters for the spillway section are determined using USACE RD-13 (Reference 23, Table 1, Page 17). For concrete gravity dams, the breach width is a multiple of the monolith widths. In this case the entire spillway section is assumed to fail. From above, the spillway section is 656 ft. in length. Therefore, the breach width is 656 ft. The side slopes are 0:1 (horizontal:vertical).

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. For the 6,660,000 cfs overtopping scenario, the difference between the headwater and tailwater is 766.7 ft. – 751.1 ft. = 15.6 ft. The difference between the headwater and breach bottom is greater than the full height of the dam, 84 ft. Therefore, the breach flow is calculated using the difference between the headwater and tailwater as follows:

$Q = (8 / 27) * 656 \text{ ft.} * (32.2 \text{ ft/sec}^2)^{0.5} * (15.6 \text{ ft.})^{1.5}$ = 67,959 cfs, rounded up to 70,000 cfs.

The total flow is assumed to be the sum of the overtopping flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 70,000 cfs + 6,660,000 cfs = 6,730,000 cfs.

For the 3,270,000 cfs overtopping scenario, the water height is 739.8 ft. – 734.2 ft. = 5.6 ft. Therefore the breach flow is calculated as follows:

$Q = (8 / 27) * 656 \text{ ft.} * (32.2 \text{ ft/sec}^2)^{0.5} * (5.6 \text{ ft.})^{1.5}$ = 14,616 cfs, rounded up to 20,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 20,000 cfs + 3,270,000 cfs = 3,290,000 cfs.

In summary, the critical potential scenarios for the De Cordova Bend Dam failure effects, including the domino-type failure from upstream dams, transposed downstream without attenuation are determined to be a spillway section breach wave height of 7.0 ft., or a spillway section total breach flow of 6,730,000 cfs. Because tailwater effects are such that the dam failure effects are minimal, the breach wave height is added to the previously determined tailwater for the controlling scenario, 626 ft. + 125.19 ft. = 751.19 ft., to determine the breach wave height flow.

The breach wave height flow is determined using FlowMaster (Reference 3) and the Manning friction formula with the downstream cross section previously identified in Table 7-22. The water surface elevation used is 751.19 ft. + 7.0 ft. = 758.19 ft. From above, the slope is 0.0004 ft./ft. and a Manning coefficient of 0.025 is applied to the channel and overbank areas. The flow for the cross section is determined to be 6,180,376 cfs. The FlowMaster results are provided in Appendix I. The breach wave height flow is less than the breach flow of 6,730,000 cfs.

The controlling dam failure scenario includes the overtopping domino-type failures of Fort Phantom Hill Dam, Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam. In addition, overtopping failure of Lake Stamford Dam is included simultaneous with the Cedar Ridge Reservoir Dam failure. The total breach flow from De Cordova Bend Dam to be transposed downstream without any attenuation to the confluence with the Paluxy River is 6,730,000 cfs. The resulting elevation at the confluence and the potential effect to CPNPP are determined by separate calculation for the evaluation of the PMF on the Squaw Creek Reservoir.

8.0 Appendices

Appendix A – Qualitative Analysis Figure and Tables – 8 Pages

Appendix B – FlowMaster Results for Hubbard Creek Dam – 3 Pages

Appendix C – FlowMaster Results for Lake Stamford Dam – 3 Pages

Appendix D – FlowMaster Results for Fort Phantom Hill Dam – 3 Pages

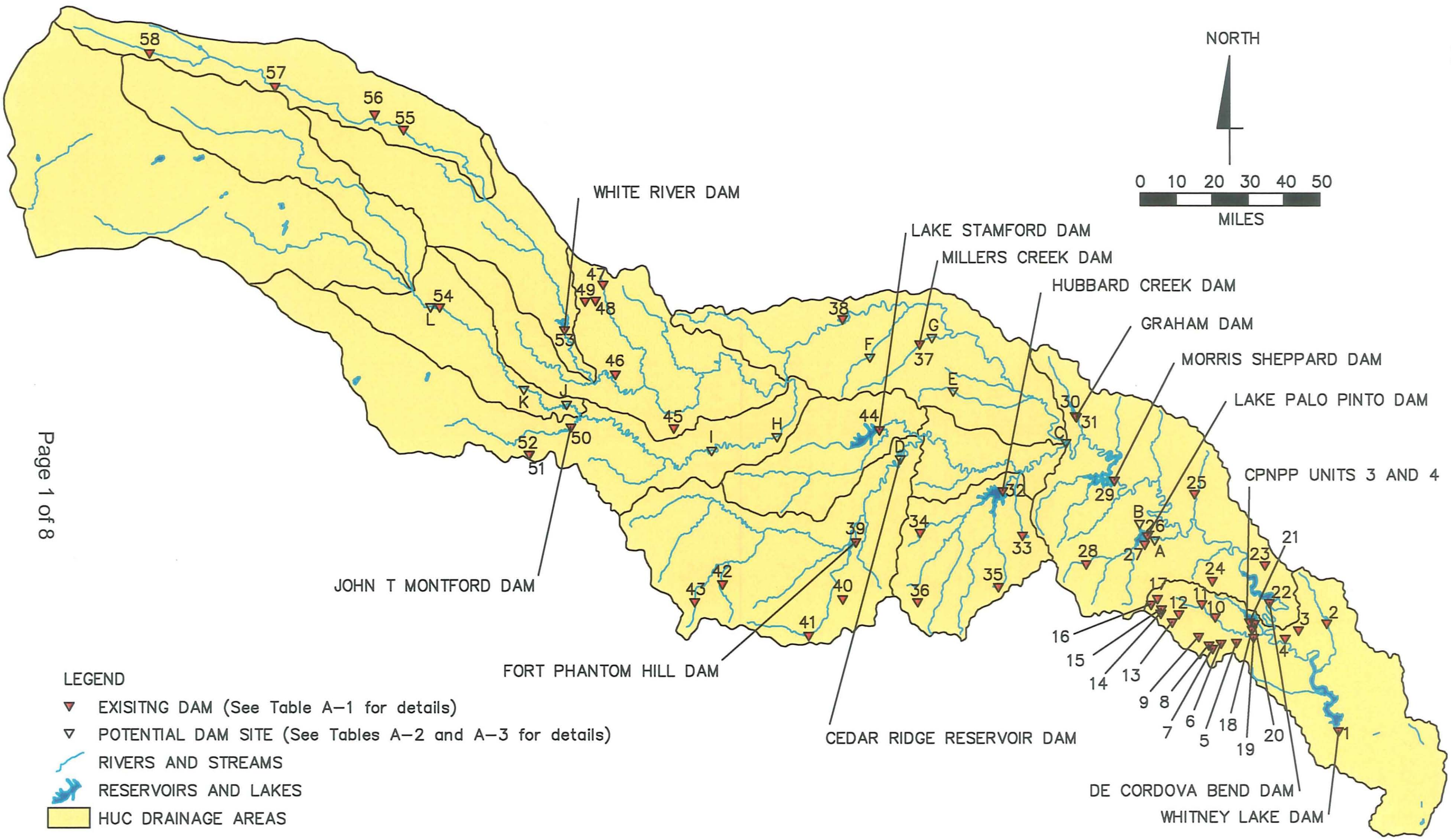
Appendix E – FlowMaster Results for Cedar Ridge Reservoir Dam – 6 Pages

Appendix F – FlowMaster Results for Morris Sheppard Dam – 6 Pages

Appendix G – FlowMaster Results for De Cordova Bend Dam Tailwater 1st Iteration – 6 Pages

Appendix H – FlowMaster Results for De Cordova Bend Dam 2nd Iteration – 3 Pages

Appendix I – FlowMaster Results for De Cordova Bend Dam Flow – 3 Pages


NORTH
0 10 20 30 40 50
MILES
WHITE RIVER DAM
LAKE STAMFORD DAM
MILLERS CREEK DAM
HUBBARD CREEK DAM
GRAHAM DAM
MORRIS SHEPPARD DAM
LAKE PALO PINTO DAM
CPNPP UNITS 3 AND 4
JOHN T MONTFORD DAM
FORT PHANTOM HILL DAM
CEDAR RIDGE RESERVOIR DAM
DE CORDOVA BEND DAM
WHITNEY LAKE DAM
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LEGEND
EXISITNG DAM (See Table A–1 for details)
POTENTIAL DAM SITE (See Tables A–2 and A–3 for details)
RIVERS AND STREAMS
RESERVOIRS AND LAKES
HUC DRAINAGE AREAS

Figure A–1

Table A-1. Information for Dams Upstream of Whitney Lake Dam

											Volume Capacity[4]	
No.	Dam Name	Owner	River	Distance (river mi.)[1]	Drainage Area (sq. mi.)	Date Completed	Type[2]	Length[3] (ft.)	Height[3] (ft.)	Surface Area (ac.)	Normal (ac.-ft.)	Maximum (ac.-ft.)
1	Whitney Lake	CESWF	Brazos River	56	17,656	1951	REPG	17,695	159	23,560	627,100	2,100,400
2	Lake Pat Cleburne Dam	City of Cleburne	Nolan River	52	100	1964	RE	5,190	78	1,550	25,600	66,700
3	Cleburne State Park Lake Dam	Texas Parks and Wildlife Department	West Fork Camp Creek	17	ns	1940	RE	1,300	62	ns	1,450	2,900
4	Lake Virginia Dam[3]	RW Leonard et al	Tr-Brazos River	11	1	1987	RE	845	56	47	898	1,169
5	Paluxy River WS SCS Site 25 Dam	Bosque SWCD	White Bluff Creek	11	11	1983	RE	2,114	60	49	200	4,485
6	Paluxy River WS SCS Site 23 Dam	Bosque SWCD	Rough Creek	18	5	1984	RE	1,260	55	22	196	1,762
7	Paluxy River WS SCS Site 21 Dam	Bosque SWCD	Lallah Branch	21	16	1982	RE	2,000	73	56	725	6,140
8	Paluxy River WS SCS Site 20 Dam	Bosque SWCD	Pony Creek	21	18	1981	RE	1,950	74	65	200	6,756
9	Paluxy River WS SCS Site 19 Dam	Bosque SWCD	Sycamore Creek	25	11	1981	RE	1,910	64	38	200	4,216
10	Paluxy River WS SCS Site 16 Dam	Brazos Valley SWCD	Goss Hollow	20	5	1980	RE	1,848	53	32	200	2,392
11	Paluxy River WS SCS Site 15 Dam	Bosque SWCD	Tr-Berry S Creek	25	12	1983	RE	1,740	55	42	236	4,064
12	Paluxy River WS SCS Site 12 Dam	Bosque SWCD	Tr-South Paluxy River	33	5	1985	RE	1,240	45	25	123	1,841
13	Paluxy River WS SCS Site 9 Dam	Bosque SWCD	Tr-South Paluxy River	36	3	1984	RE	920	45	20	164	1,107

											Volume Capacity[4]	
No.	Dam Name	Owner	River	Distance (river mi.)[1]	Drainage Area (sq. mi.)	Date Completed	Type[2]	Length[3] (ft.)	Height[3] (ft.)	Surface Area (ac.)	Normal (ac.-ft.)	Maximum (ac.-ft.)
14	Paluxy River WS SCS Site 3 Dam	Bosque SWCD	Tr-Paluxy River	39	2	1987	RE	865	51	16	110	821
15	Paluxy River WS SCS Site 6 Dam	Bosque SWCD	Straight Creek	38	5	1980	RE	1,168	53	41	150	1,211
16	Paluxy River WS SCS Site 1 Dam	Bosque SWCD	Tr-North Paluxy River	40	4	1984	RE	850	54	24	160	1,512
17	Paluxy River WS SCS Site 5 Dam	Bosque SWCD	Germany Creek	39	160	1988	RE	1,640	58	25	171	1,604
18	Wheeler Branch Dam[5]	Somervell County Water District	Wheeler Branch	5	1.6	2007	RE	1,750	80	180	4,118	4,118
19	Paluxy River Channel Dam[5]	Somervell County Water District	Paluxy River	3	428	2007	PG	ns	8	9	35	35
20	Squaw Creek Dam	TU Electric	Squaw Creek	5	64	1977	RE	4,690	152	3,228	151,047	199,427
21	Safe Shutdown Impoundment Dam	TU Electric	Panther Branch	6	7	1977	ER	1,520	70	7	367	900
22	De Cordova Bend Dam	Brazos River Authority	Brazos River	33	15,451	1969	PG	2,200	79	1,350	136,823	240,640
23	Ruckers Creek WS SCS Site 1 Dam	Brazos Valley SWCD	Rucker Creek	49	6	1968	RE	2,080	50	33	133	2,375
24	Star Hollow Lake Dam	Bank One Texas NA Trustee JM Leonard Trust	Star Hollow Creek	84	ns	1967	RE	1,120	54	92	1,454	1,959
25	Lake Mineral Wells Dam	City of Mineral Wells	Rock Creek	91	63	1920	RE	1,760	70	668	7,065	16,356

											Volume Capacity[4]	
No.	Dam Name	Owner	River	Distance (river mi.)[1]	Drainage Area (sq. mi.)	Date Completed	Type[2]	Length[3] (ft.)	Height[3] (ft.)	Surface Area (ac.)	Normal (ac.-ft.)	Maximum (ac.-ft.)
26	Lake Palo Pinto Dam	Palo Pinto County MWD No 1	Palo Pinto Creek	104	471	1964	RE	1,255	93	2,661	44,100	170,735
27	Waddell Ranch Dam No 3	Earl Waddell Inc	Joes Creek	110	ns	1975	RE	613	54	16	307	488
28	Lake Tucker Dam	City of Strawn	Russell Creek	126	24	1937	RE	900	97	81	1,600	2,500
29	Morris Sheppard	Brazos River Authority	Brazos R	162	13,310	1941	CB	2,740	154	17,624	556,220	556,220
30	Graham Dam	City of Graham	Salt Creek	219	42	1958	RE	4,300	82	1,900	39,000	105,000
31	Eddleman Dam	City of Graham	Flint Creek	218	42	1929	RE	4,495	57	650	13,386	35,000
32	Hubbard Creek Dam	West Central Texas MUD	Hubbard Creek	261	1,107	1962	RE	12,580	109	15,250	317,750	720,000
33	Gonzales Creek Dam	City of Breckenridge	Gonzales Creek	271	115	1948	RE	2,700	50	954	11,400	38,242
34	McCarty Lake Dam	City of Albany	Salt Prong Hubbard Creek	290	44	1942	RE	1,250	50	263	2,600	6,696
35	Williamson Dam	City of Cisco	Sandy Creek	292	26	1923	CB	1,064	96	1,817	45,000	45,000
36	Mexia Dam	City of Baird	Mexia Creek	307	ns	1950	RE	1,660	52	ns	2,070	3,370
37	Millers Creek Dam	North Cent Tex MWA et al	Millers Creek	305	ns	1974	RE	8,000	75	2,882	29,171	131,000
38	Lake Davis Dam	Eagle Ranch Inc	Dutchman Creek	347	ns	1959	RE	6,864	32	ns	5,395	19,000
39	Fort Phantom Hill Dam	City of Abilene	Big Elm Creek	375	463	1938	RE	3,800	84	4,246	70,036	127,000
40	Lake Kirby Dam	City of Abilene	Cedar Creek	399	42	1928	RE	4,200	50	780	7,620	17,811
41	Lake Abilene Dam	City of Abilene	Elm Creek	409	101	1921	RE	5,040	64	583	45,000	45,000
42	Lake Sweetwater Dam	City of Sweetwater	Bitter Creek	429	104	1930	RE	3,030	58	221	2,544	19,340
43	Lake Trammel Dam	City of Sweetwater	Sweetwater Creek	439	49	1915	RE	1,160	59	160	2,500	5,890

											Volume Capacity[4]	
No.	Dam Name	Owner	River	Distance (river mi.)[1]	Drainage Area (sq. mi.)	Date Completed	Type[2]	Length[3] (ft.)	Height[3] (ft.)	Surface Area (ac.)	Normal (ac.-ft.)	Maximum (ac.-ft.)
44	Lake Stamford Dam	City of Stamford	Paint Creek	332	360	1953	RE	6,600	71	4,690	57,927	150,000
45	So Relle Lake Dam	Relle	Stinking Creek	453	ns	1964	RE	1,000	50	40	412	1,000
46	Hagins Panther Canyon Lake Dam	Hagins	Tr-Salt Fork Brazos River	483	ns	1969	RE	300	50	10	140	320
47	Duck Creek WS SCS Site 1 Dam	Dickens County WCID No 1	Duck Creek	502	20	1968	RE	3,600	62	79	634	10,750
48	Duck Creek WS SCS Site 5 Dam	Dickens County WCID No 1	Cottonwood Creek	500	22	1969	RE	2,550	71	148	2,249	7,900
49	Duck Creek WS SCS Site 7 Dam	Dickens County WCID No 1	Dockum Creek	502	12	1968	RE	2,900	61	33	200	4,712
50	John T Montford Dam	Brazos River Authority	Double Mountain Fork Brazos R	513	394	1994	RE	440	141	2,884	115,937	354,500
51	Parks Lake Dam	Parks	Tr-Griffin Creek	539	ns	1971	RE	1,142	50	6	110	220
52	Big Tank Dam	Parks	Tr- Double Mtn Fk Brazos River	539	ns	1965	RE	600	65	ns	185	490
53	White River Dam	White River Municipal Water District	White River	518	172	1963	RE	4,400	80	1,477	31,537	80,000
54	McMillan Dam	Lubbock County WCID No 1	Double Mountain Fork Brazos R	577	236	1960	RE	1,600	76	200	4,200	8,280
55	Lower Running Water Draw WS SCS Site 3 Dam	Hale County SWCD	Running Water Draw	606	390	1982	RE	2,500	37	54	8,213	14,312

											Volume Capacity[4]	
No.	Dam Name	Owner	River	Distance (river mi.)[1]	Drainage Area (sq. mi.)	Date Completed	Type[2]	Length[3] (ft.)	Height[3] (ft.)	Surface Area (ac.)	Normal (ac.-ft.)	Maximum (ac.-ft.)
56	Lower Running Water Draw WS SCS Site 2 Dam	Hale County SWCD	N Fork Running Water Draw	618	30	1977	RE	3,430	41	42	5,429	7,383
57	Running Water Draw WS SCS Site 3 Dam	Parmer County SWCD	Running Water Draw	649	124	1979	RE	3,250	55	233	4,427	18,499
58	Running Water Draw Site 1 Dam	Central Curry Soil and Water Conservation District	Running Water Draw	692	128	1975	RE	3,208	65	1,581	2,170	25,150

Information obtained from National Atlas unless otherwise noted.

ns = not specified

1. Distance in river miles from the dam to the confluence of the Brazos River and Paluxy River.
2. Type of dam:
   - RE = Earth
   - ER = Rockfill
   - PG = Gravity
   - CB = Buttress
3. Information obtained from the U.S. Army Corps of Engineers National Inventory of Dams database.
4. Normal storage is the total storage below the normal retention level, including dead and inactive storage and excluding any flood control or surcharge storage. Maximum storage is the total storage below the maximum attainable water surface elevation, including any surcharge storage.
5. Information obtained from Somervell County Water District and the 2011 Brazos G Regional Water Plan.

Table A-2. Information from the 2011 Brazos G Regional Water Plan for Strategies Upstream of Whitney Lake Dam

No.	Strategy Name	Status[1]	River	Distance (river mi.)[2]	Drainage Area (sq. mi.)	Type[3]	Length (ft.)	Height (ft.)	Surface Area (ac.)	Volume Capacity (ac.-ft.)
A	Turkey Peak Reservoir	R	Palo Pinto Creek	101	ns	ns	ns	ns	648	22,577
B	Lake Palo Pinto Off-Channel Reservoir	I	Wilson Hollow	109	ns	RE	1,550	ns	182	10,000 up to 22,000
C	South Bend Reservoir	I	Brazos River	228	13,168	RE	14,784	ns	29,877	771,604
D	Cedar Ridge Reservoir	R	Clear Fork of the Brazos River	334	2,748	ns	ns	ns	6,635	227,127
E	Throckmorton Reservoir	I	North Elm Creek	278	82	ns	ns	ns	1,161	15,900
F	Millers Creek Reservoir Augmentation	R	Lake Creek	337	ns	RE	5,000	8	360	ns
G	Millers Creek Reservoir Augmentation	R	Millers Creek	301	292	RE	ns	ns	2,541	46,645
H	Double Mountain Fork East Reservoir	I	Double Mountain Fork of the Brazos River	403	1,937	ns	ns	ns	10,814	280,814
I	Double Mountain Fork West Reservoir	I	Double Mountain Fork of the Brazos River	433	1,669	ns	ns	ns	6,632	215,254

ns = not specified
1. Status of water management strategy in the 2011 Brazos G Regional Water Plan
   I = identified as potentially feasible water management strategy
   R = recommended water management strategy
2. Distance in river miles from the dam to the confluence of the Brazos River and Paluxy River.
3. Type of dam: RE = Earth

Table A-3. Information from the Llano Estacado Regional Water Plan for Strategies Upstream of Whitney Lake Dam

No.	Strategy Name	River	Distance (river mi.)[1]	Drainage Area (sq. mi.)	Type[2]	Length (ft.)	Height (ft.)	Surface Area (ac.)	Volume Capacity (ac.-ft.)
J	Diversion Reservoir	North Fork Double Mountain Fork Brazos River	515	ns	ns	ns	ns	ns	1,000
K	Post Reservoir	North Fork Double Mountain Fork Brazos River	536	ns	RE	5,800	ns	2,280	56,000
L	Lake 7	North Fork Double Mountain Fork Brazos River	580	ns	ns	ns	ns	ns	20,700

ns = not specified
1. Distance in river miles from the dam to the confluence of the Brazos River and Paluxy River.
2. Type of dam: RE = Earth

Hubbard Creek - Tailwater Elevation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00051	ft/ft
Discharge	630000.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-42+72	1188.00
-38+67	1180.00
-36+31	1170.00
-34+65	1160.00
-32+85	1150.00
-30+78	1140.00
-23+54	1130.00
-18+98	1120.00
-16+40	1110.00
-1+14	1110.00
-0+95	1100.00
-0+75	1090.00
-0+34	1087.00
0+34	1087.00
0+59	1090.00
0+95	1100.00
1+40	1110.00
25+04	1120.00
25+35	1130.00
26+11	1140.00
26+56	1150.00
27+66	1160.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient

Hubbard Creek - Tailwater Elevation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-42+72, 1188.00)	(27+66, 1160.00)	0.025

Results

Normal Depth	41.72	ft
Elevation Range	1087.00 to 1188.00 ft	
Flow Area	75280.86	ft²
Wetted Perimeter	4835.84	ft
Top Width	4826.57	ft
Normal Depth	41.72	ft
Critical Depth	33.66	ft
Critical Slope	0.00446	ft/ft
Velocity	8.37	ft/s
Velocity Head	1.09	ft
Specific Energy	42.81	ft
Froude Number	0.37	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	41.72	ft
Critical Depth	33.66	ft
Channel Slope	0.00051	ft/ft
Critical Slope	0.00446	ft/ft

Cross Section - Hubbard Creek Tailwater

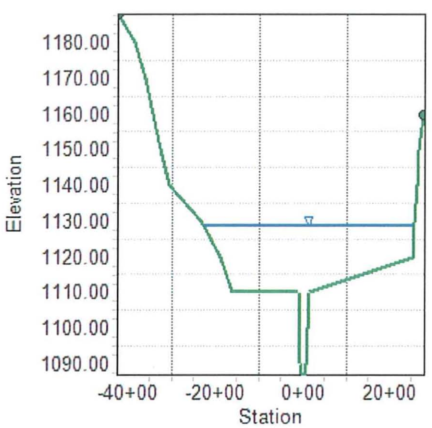
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00051	ft/ft
Normal Depth	41.72	ft
Discharge	630000.00	ft³/s

Cross Section Image



ake Sta ord - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00140	ft/ft
Discharge	350000.00	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
-2+54	1450.00
-1+34	1400.00
-0+43	1370.00
0+00	1364.00
0+31	1370.00
3+25	1380.00
3+85	1390.00
4+48	1400.00
5+68	1410.00
8+12	1420.00
10+13	1430.00
12+11	1440.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-2+54, 1450.00)	(12+11, 1440.00)	0.025

Results

Normal Depth	45.11	ft
Elevation Range	1364.00 to 1450.00 ft	
Flow Area	18273.92	ft^2
Wetted Perimeter	723.03	ft
Top Width	713.23	ft
Normal Depth	45.11	ft

ake Sta ord - Tailwater le ation

Results

Critical Depth		37.37	ft
Critical Slope		0.00331	ft/ft
Velocity		19.15	ft/s
Velocity Head		5.70	ft
Specific Energy		50.81	ft
Froude Number		0.67	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	45.11	ft
Critical Depth	37.37	ft
Channel Slope	0.00140	ft/ft
Critical Slope	0.00331	ft/ft

Cross Section - ake Sta ord Tailwater le ation

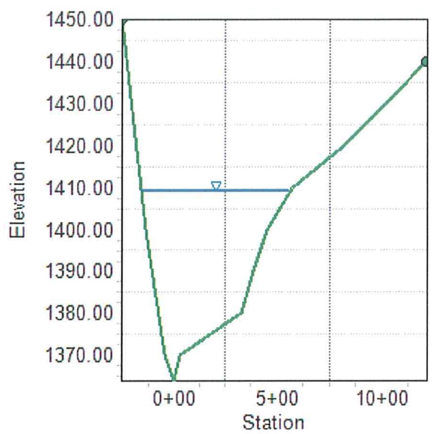
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00140	ft/ft
Normal Depth	45.11	ft
Discharge	350000.00	ft³/s

Cross Section Image



ort anto Hill - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00052	ft/ft
Discharge	410000.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-21+73	1620.00
-19+08	1610.00
-17+43	1600.00
-16+00	1590.00
-14+87	1580.00
-13+61	1570.00
-2+72	1560.00
-2+10	1550.00
-0+53	1540.00
0+00	1538.00
0+71	1540.00
1+12	1560.00
9+68	1565.00
29+04	1570.00
35+94	1580.00
36+59	1590.00
40+08	1600.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-21+73, 1620.00)	(40+08, 1600.00)	0.025

Results

Normal Depth	38.95	ft

ort anto Hill - Tailwater le ation

Results

Elevation Range	1538.00 to 1620.00 ft	
Flow Area	57853.21	ft^2
Wetted Perimeter	4838.60	ft
Top Width	4832.40	ft
Normal Depth	38.95	ft
Critical Depth	32.47	ft
Critical Slope	0.00487	ft/ft
Velocity	7.09	ft/s
Velocity Head	0.78	ft
Specific Energy	39.73	ft
Froude Number	0.36	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	38.95	ft
Critical Depth	32.47	ft
Channel Slope	0.00052	ft/ft
Critical Slope	0.00487	ft/ft

Cross Section - ort anto Hill Tailwater

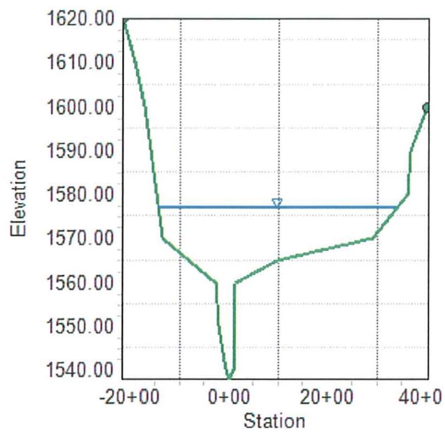
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00052	ft/ft
Normal Depth	38.95	ft
Discharge	410000.00	ft³/s

Cross Section Image



Cedar id e - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00090	ft/ft
Discharge	1160000.00	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
-22+88	1480.00
-18+63	1470.00
-16+97	1460.00
-15+86	1450.00
-14+67	1440.00
-11+26	1430.00
-9+78	1420.00
-7+57	1410.00
-5+78	1400.00
-2+49	1390.00
-1+18	1380.00
-0+84	1370.00
-0+53	1360.00
-0+31	1356.00
0+36	1356.00
0+50	1360.00
1+45	1400.00
2+14	1450.00
2+52	1460.00
3+16	1470.00
4+29	1500.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient

Cedar id e - Tailwater le ation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-22+88, 1480.00)	(4+29, 1500.00)	0.025

Results

Normal Depth		85.71	ft
Elevation Range	1356.00 to 1500.00 ft		
Flow Area		60517.92	ft²
Wetted Perimeter		1717.07	ft
Top Width		1689.94	ft
Normal Depth		85.71	ft
Critical Depth		69.76	ft
Critical Slope		0.00300	ft/ft
Velocity		19.17	ft/s
Velocity Head		5.71	ft
Specific Energy		91.42	ft
Froude Number		0.56	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	85.71	ft
Critical Depth	69.76	ft
Channel Slope	0.00090	ft/ft
Critical Slope	0.00300	ft/ft

Cross Section - Cedar id eTailwater le ation

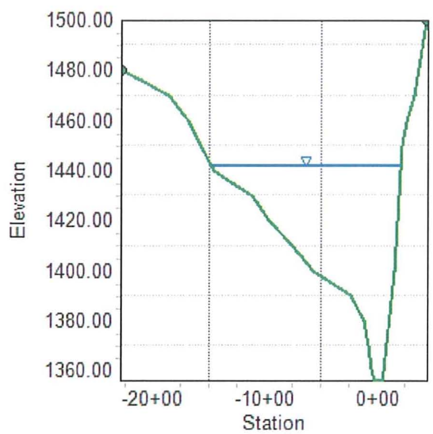
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00090	ft/ft
Normal Depth	85.71	ft
Discharge	1160000.00	ft³/s

Cross Section Image



Cedar id e - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00090	ft/ft
Discharge	2900000.00	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
-22+88	1480.00
-18+63	1470.00
-16+97	1460.00
-15+86	1450.00
-14+67	1440.00
-11+26	1430.00
-9+78	1420.00
-7+57	1410.00
-5+78	1400.00
-2+49	1390.00
-1+18	1380.00
-0+84	1370.00
-0+53	1360.00
-0+31	1356.00
0+36	1356.00
0+50	1360.00
1+45	1400.00
2+14	1450.00
2+52	1460.00
3+16	1470.00
4+29	1500.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient

Cedar id e - Tailwater Elevation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-22+88, 1480.00)	(4+29, 1500.00)	0.025

Results

Normal Depth	115.35	ft
Elevation Range	1356.00 to 1500.00 ft	
Flow Area	117357.88	ft²
Wetted Perimeter	2274.83	ft
Top Width	2241.65	ft
Normal Depth	115.35	ft
Critical Depth	95.75	ft
Critical Slope	0.00266	ft/ft
Velocity	24.71	ft/s
Velocity Head	9.49	ft
Specific Energy	124.84	ft
Froude Number	0.60	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	115.35	ft
Critical Depth	95.75	ft
Channel Slope	0.00090	ft/ft
Critical Slope	0.00266	ft/ft

Cross Section - Cedar id e Tailwater le ation

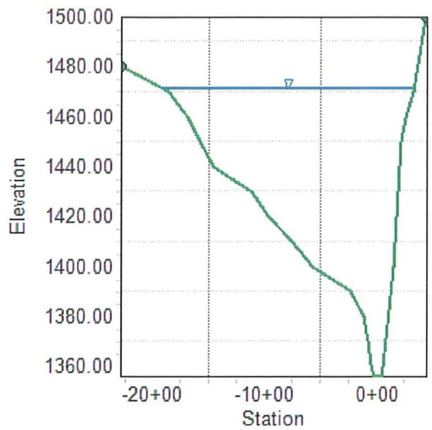
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00090	ft/ft
Normal Depth	115.35	ft
Discharge	2900000.00	ft³/s

Cross Section Image



orris S e ard - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00160	ft/ft
Discharge	5420000.00	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
-11+75	1050.00
-7+44	1000.00
-6+51	950.00
-6+03	940.00
-5+62	930.00
-5+18	920.00
-4+05	910.00
-3+85	900.00
-3+33	880.00
-2+66	870.00
3+10	870.00
4+40	880.00
4+95	890.00
5+47	900.00
6+56	920.00
10+68	930.00
11+64	940.00
12+15	950.00
12+77	960.00
13+32	970.00
14+25	980.00
15+05	990.00
15+89	1000.00

Roughness Segment Definitions

orris S e ard - Tailwater le ation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-11+75, 1050.00)	(15+89, 1000.00)	0.025

Results

Normal Depth	103.01	ft
Elevation Range	870.00 to 1050.00 ft	
Flow Area	138600.60	ft²
Wetted Perimeter	2077.72	ft
Top Width	2053.78	ft
Normal Depth	103.01	ft
Critical Depth	94.69	ft
Critical Slope	0.00234	ft/ft
Velocity	39.11	ft/s
Velocity Head	23.76	ft
Specific Energy	126.77	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	103.01	ft
Critical Depth	94.69	ft
Channel Slope	0.00160	ft/ft
Critical Slope	0.00234	ft/ft

Cross Section - orris S e ard Tailwater

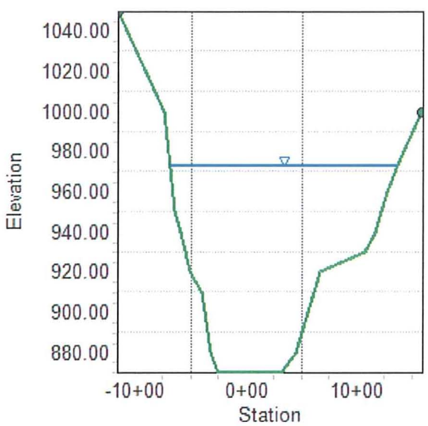
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00160	ft/ft
Normal Depth	103.01	ft
Discharge	5420000.00	ft³/s

Cross Section Image



orris S e ard - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00160	ft/ft
Discharge	5120000.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-11+75	1050.00
-7+44	1000.00
-6+51	950.00
-6+03	940.00
-5+62	930.00
-5+18	920.00
-4+05	910.00
-3+85	900.00
-3+33	880.00
-2+66	870.00
3+10	870.00
4+40	880.00
4+95	890.00
5+47	900.00
6+56	920.00
10+68	930.00
11+64	940.00
12+15	950.00
12+77	960.00
13+32	970.00
14+25	980.00
15+05	990.00
15+89	1000.00

Roughness Segment Definitions

orris S e ard - Tailwater le ation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-11+75, 1050.00)	(15+89, 1000.00)	0.025

Results

Normal Depth	100.34	ft
Elevation Range	870.00 to 1050.00 ft	
Flow Area	133151.68	ft²
Wetted Perimeter	2047.08	ft
Top Width	2023.95	ft
Normal Depth	100.34	ft
Critical Depth	92.20	ft
Critical Slope	0.00236	ft/ft
Velocity	38.45	ft/s
Velocity Head	22.98	ft
Specific Energy	123.31	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	100.34	ft
Critical Depth	92.20	ft
Channel Slope	0.00160	ft/ft
Critical Slope	0.00236	ft/ft

Cross Section - orris S e ard Tailwater

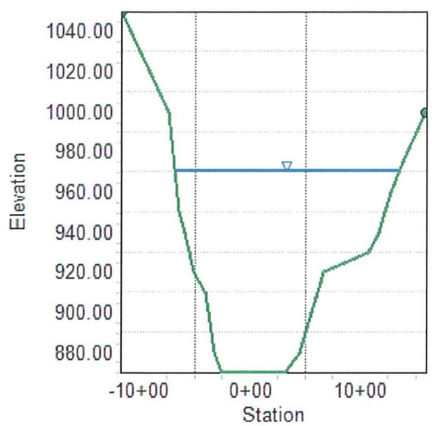
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00160	ft/ft
Normal Depth	100.34	ft
Discharge	5120000.00	ft³/s

Cross Section Image



e Cordo a end - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00040	ft/ft
Discharge	5000000.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-66+74	760.00
-64+81	750.00
-44+25	740.00
-35+28	730.00
-29+42	720.00
-26+23	710.00
-23+88	700.00
-17+77	690.00
-13+10	680.00
-7+67	670.00
-5+84	660.00
-4+66	650.00
-1+87	640.00
-1+50	630.00
-1+13	626.00
1+40	626.00
1+67	630.00
2+43	650.00
3+41	700.00
5+43	710.00
7+76	720.00
19+14	730.00
30+47	740.00

Roughness Segment Definitions

e Cordo a end - Tailwater le ation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-66+74, 760.00)	(30+47, 740.00)	0.025

Results

Normal Depth	126.79	ft
Elevation Range	626.00 to 760.00 ft	
Flow Area	369386.29	ft²
Wetted Perimeter	9613.44	ft
Top Width	9581.89	ft
Normal Depth	126.79	ft
Critical Depth	92.34	ft
Critical Slope	0.00270	ft/ft
Velocity	13.54	ft/s
Velocity Head	2.85	ft
Specific Energy	129.64	ft
Froude Number	0.38	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	126.79	ft
Critical Depth	92.34	ft
Channel Slope	0.00040	ft/ft
Critical Slope	0.00270	ft/ft

Cross Section - e Cordo a end Tailwater

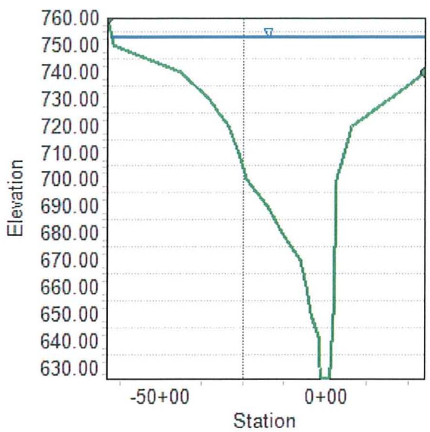
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00040	ft/ft
Normal Depth	126.79	ft
Discharge	5000000.00	ft³/s

Cross Section Image



e Cordo a end - Tailwater le ation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00040	ft/ft
Discharge	2750000.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-66+74	760.00
-64+81	750.00
-44+25	740.00
-35+28	730.00
-29+42	720.00
-26+23	710.00
-23+88	700.00
-17+77	690.00
-13+10	680.00
-7+67	670.00
-5+84	660.00
-4+66	650.00
-1+87	640.00
-1+50	630.00
-1+13	626.00
1+40	626.00
1+67	630.00
2+43	650.00
3+41	700.00
5+43	710.00
7+76	720.00
19+14	730.00
30+47	740.00

Roughness Segment Definitions

e Cordo a end - Tailwater le ation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-66+74, 760.00)	(30+47, 740.00)	0.025

Results

Normal Depth	108.29	ft
Elevation Range	626.00 to 760.00 ft	
Flow Area	218348.35	ft²
Wetted Perimeter	6331.45	ft
Top Width	6312.84	ft
Normal Depth	108.29	ft
Critical Depth	75.22	ft
Critical Slope	0.00292	ft/ft
Velocity	12.59	ft/s
Velocity Head	2.47	ft
Specific Energy	110.75	ft
Froude Number	0.38	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	108.29	ft
Critical Depth	75.22	ft
Channel Slope	0.00040	ft/ft
Critical Slope	0.00292	ft/ft

Cross Section - e Cordo a end Tailwater

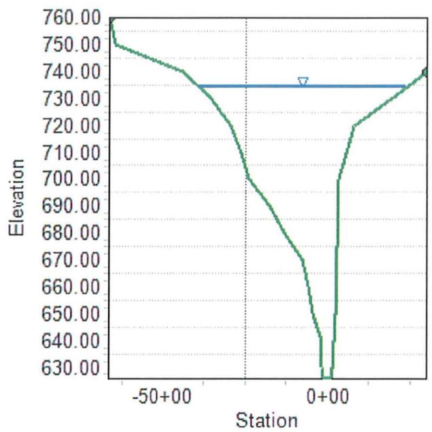
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00040	ft/ft
Normal Depth	108.29	ft
Discharge	2750000.00	ft³/s

Cross Section Image



e Cordova end - Tailwater Elevation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00040	ft/ft
Discharge	4670000.00	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
-66+74	760.00
-64+81	750.00
-44+25	740.00
-35+28	730.00
-29+42	720.00
-26+23	710.00
-23+88	700.00
-17+77	690.00
-13+10	680.00
-7+67	670.00
-5+84	660.00
-4+66	650.00
-1+87	640.00
-1+50	630.00
-1+13	626.00
1+40	626.00
1+67	630.00
2+43	650.00
3+41	700.00
5+43	710.00
7+76	720.00
19+14	730.00
30+47	740.00

Roughness Segment Definitions

e Cordova end - Tailwater Elevation

Input Data

Start Station	Ending Station	Roughness Coefficient
(-66+74, 760.00)	(30+47, 740.00)	0.025

Results

Normal Depth	125.19	ft
Elevation Range	626.00 to 760.00 ft	
Flow Area	354080.35	ft²
Wetted Perimeter	9580.92	ft
Top Width	9551.01	ft
Normal Depth	125.19	ft
Critical Depth	90.15	ft
Critical Slope	0.00273	ft/ft
Velocity	13.19	ft/s
Velocity Head	2.70	ft
Specific Energy	127.90	ft
Froude Number	0.38	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	125.19	ft
Critical Depth	90.15	ft
Channel Slope	0.00040	ft/ft
Critical Slope	0.00273	ft/ft

Cross Section - e Cordo a end Tailwater

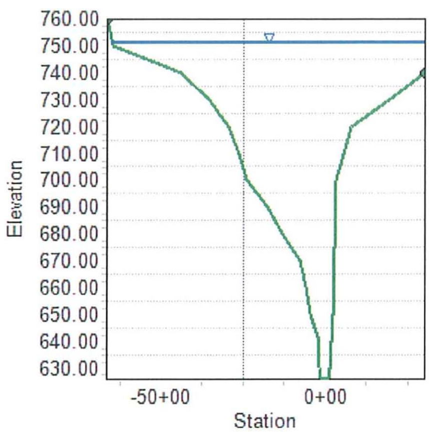
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00040	ft/ft
Normal Depth	125.19	ft
Discharge	4670000.00	ft³/s

Cross Section Image



e Cordo a end - Tailwater

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.00040	ft/ft
Normal Depth	132.19	ft

Section Definitions

Station (ft)	Elevation (ft)
-66+74	760.00
-64+81	750.00
-44+25	740.00
-35+28	730.00
-29+42	720.00
-26+23	710.00
-23+88	700.00
-17+77	690.00
-13+10	680.00
-7+67	670.00
-5+84	660.00
-4+66	650.00
-1+87	640.00
-1+50	630.00
-1+13	626.00
1+40	626.00
1+67	630.00
2+43	650.00
3+41	700.00
5+43	710.00
7+76	720.00
19+14	730.00
30+47	740.00

Roughness Segment Definitions

e Cordo a end - Tailwater

Input Data

Start Station	Ending Station	Roughness Coefficient
(-66+74, 760.00)	(30+47, 740.00)	0.025

Results

Discharge	6180376.16	ft³/s
Elevation Range	626.00 to 760.00 ft	
Flow Area	421386.60	ft²
Wetted Perimeter	9723.15	ft
Top Width	9686.07	ft
Normal Depth	132.19	ft
Critical Depth	101.88	ft
Critical Slope	0.00278	ft/ft
Velocity	14.67	ft/s
Velocity Head	3.34	ft
Specific Energy	135.53	ft
Froude Number	0.39	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	132.19	ft
Critical Depth	101.88	ft
Channel Slope	0.00040	ft/ft
Critical Slope	0.00278	ft/ft

Cross Section - e Cordo a end Tailwater

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.00040	ft/ft
Normal Depth	132.19	ft
Discharge	6180376.16	ft³/s

Cross Section Image